R. H. MANN.

The Association

of

Engineering and Shipbuilding Draughtsmen.

THE USE OF STEEL TUBES IN STRUCTURAL DESIGN.

By F. H. ABRAHAMS, A.M.I.Struct.E., M.Inst.W.

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SYNOPSIS.

Introduction — Examples — General Considerations — Compressive Stresses — Tension — Bending — Torsion — General Procedure — Properties of Tubes — Formulae recommendations — Types of Connections — Comparative Designs — Roof Trusses — Lattice Girders — Stanchions — Footbridge — Tables giving Properties of Sections, Working Stresses and Safe Loads.

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THE USE OF STEEL TUBES IN STRUCTURAL DESIGN.

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INTRODUCTION.

DEVELOPMENTS in welded structural steelwork in recent years have drawn increasing attention to the inadequacy of standard rolled steel sections to meet present-day requirements. These sections, almost without exception, were devised to suit rivetted construction, and necessarily have no particular relationship to the requirements for welded work.

Designers of welded frames are thus faced with the choice of using sections which may be unsuitable and uneconomical for the purpose required, or alternatively, of building up composite sections with flats, plates, split joists, etc., or any other combination that can be devised.

Whichever course is adopted, designers are obviously at a serious disadvantage, and there can be little doubt that this difficulty has to some extent retarded the extensive adoption of welded structures.

It seems likely that as welding technique and procedure become more stabilised, and industrial conditions generally become more settled, entirely new shapes will be evolved to suit welding requirements, or at least that some modification will be made to existing profiles for this purpose. In the meantime designers must make the best use possible of existing facilities.

In this pamphlet the author wishes to draw attention to the fairly wide possibilities of tubular steel sections as a medium for design in connection with welded structural frameworks. The term "structural frameworks" is used here in its usually accepted sense as applying to such structures as framed buildings, bridges, and similar work commonly dealt with in the structural drawing office. Items such as aircraft frames, rolling stock, and so on are therefore excluded.

Tubes have, of course, been employed for many years as subsidiary parts of structures. A common use which will at once leap to the mind is the case of lacing bars in bridge truss members. The main objection to using tubes for such a purpose in the past has been that the ends of each bar had to be flattened for rivetting in position, a proceeding which often made the cost prohibitive, unless a large number all alike were required.

Tubes with screwed joints have also been used for light frames carrying small loads, such as storage racks, sheeted huts and shelters. In addition, flanged pipes which for design purposes come within the category of tubes have been employed extensively as piles for piers and jetties, for which purpose they are eminently suitable.

Potentialities of tubular construction are most strikingly illustrated in the case of the Forth Bridge, which, although built fifty years ago and designed for smaller loads than more modern bridges, still gives excellent service, and is indeed one of the wonders of the structural world. The main compression members of this bridge are constructed of built-up rivetted bent plates, varying in diameter from 12′-0″ to 6′-6″, and although this type of construction does not come strictly within the limits dealt with in this pamphlet, it does at least indicate the general possibilities of tubular construction.

Recent developments in welding appear to call for a more extensive consideration of the use of tubular sections in braced frame construction generally, and in the following notes the author examines this question in some detail and throws out various suggestions, as much for the purpose of stimulating thought and discussion as for seeking to supersede more normal procedure.

To date tubular construction has not been used or even considered to any great extent in this country, except by a few firms specialising in this class of work, and very little British literature appears to be available on the subject. Numerous examples of tubular frames of quite considerable size can, however, be found in other countries.

Probably the most notable example in Europe is the Littoria Observation Tower, built in 1933 for the Milan Art Exhibition, the general construction of which is shown in Figs. 1 and 2.

This structure is of hexagonal shape, and about 350′-0″ high. The sides vary in width from nearly 20′-0″ at the base to nearly 15′-0″ at the top, and the main framework consists of six corner posts, braced together vertically and horizontally. Inside this outer frame and braced to it, is a similar hexagonal structure supporting the lift and spiral staircase. The structure is built completely with seamless tubes. Main posts are of high tensile steel, varying in diameter from 17″ at the base to 6½″ at the top, and in thickness from $\frac{5}{8}$″ to $\frac{5}{16}$″. Bracings vary from 7″ diameter near the base to 4¼″ diameter near the top, and in thickness from $\frac{3}{8}$″ to ¼″. Joints are made with gussets inserted in slots in the main posts. Ends of all bracing members are forked, bent together, slid over the gussets, and welded thereto, thus completely sealing the ends and obviating any possibility of internal corrosion. Vertical gussets passing through the main posts are shaped dovetail fashion, so that each plate can be inserted at least partially through

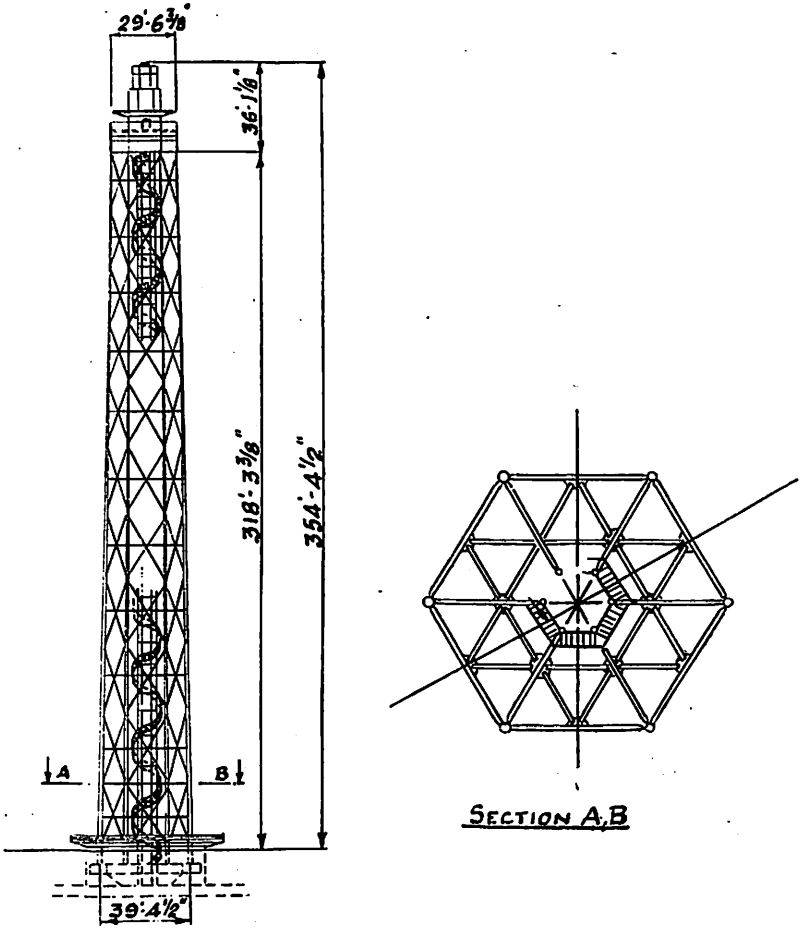


Fig. 1—Littoria Observation Tower, Milan.

both sides of the tube with continuous welds on the near side and short welds on the far side. Service bolts were used to hold the bracings in position during site welding. The main posts are jointed clear of the bracing intersections by means of flanges bolted together. Total weight of steel in this structure is about 150 tons, and is very much less than would have been the case with a normal framed rivetted tower.

So satisfactory was this tower found to be in all respects that a similar tower was built for the Italian Pavilion at the Brussels Exhibition in 1935.

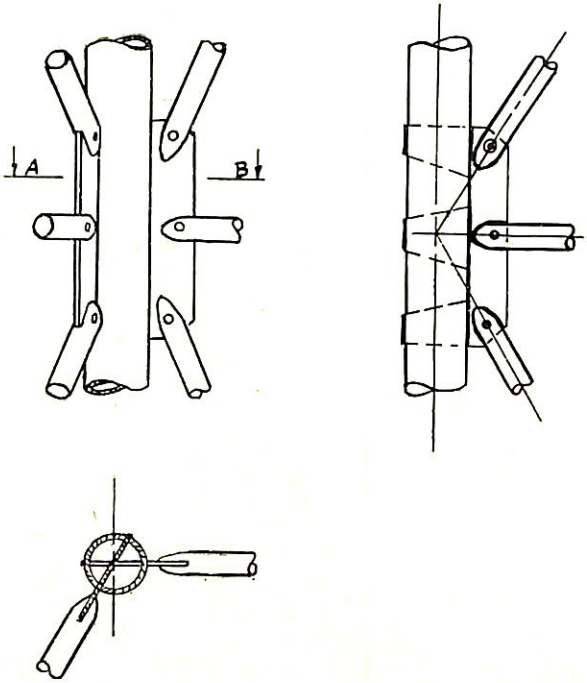


Fig. 2—Details of Joints for Littoria Observation Tower.

Further examples of tubular tower construction are provided by the four Illumination Towers at the Strahov Stadium, Prague, which were installed for evening sports festivals.

These towers are truncated square pyramids in shape, the base being about 6′-9″ square and the top about 2′-6″ square. The total height is about 100′-0″, divided into 15 panels varying from 9′-0″ high near the base to 4′-9″ at the top. On top of the towers is a platform about 9′-3″ × 5′-0″, supporting the battery of lights. The towers are of rigid frame (Veirendeel) construction, the joints being stiffened with vertical and horizontal gussets to provide the necessary rigidity. Seamless tubes are used throughout for all members. The posts are of high tensile steel, varying in diameter from 5″ at the base to 3″ at the top. Horizontal struts are of mild steel, 5½″ diameter near the base and 2½″ diameter at the top. The towers were shop-welded in two sections, site joints being made with sleeves and the two halves being assembled with service bolts before site welding.

Lattice tubular construction has also been used for a transmission mast, 130′-0″ high at Ghent Radio Station, Belgium. This mast is triangular in shape, one metre wide at mid-height and tapering towards the bottom and top. The tubes are 2″ diameter and 1/10″ thick. The mast is made in three sections, all shop welded, and sleeves are used for the site joints.

Tubular masts have been used extensively throughout Czecho-Slovakia for electricity transmission lines, for which purpose they are eminently suitable, and much more satisfactory than normal English construction with a multiplicity of light bracing members. The masts are 73′-0″ high and spaced about 1000′-0″ apart. They are square in plan, with all members of $3\frac{1}{2}$″ diameter high tensile seamless tubes about $\frac{1}{8}$″ thick. With bracings of the same section as the legs, the bracing ends are flared out to an elliptical shape to make a good joint. Short sleeves are provided to the posts at the joints to facilitate fabrication, the sleeves being in halves with joints at an angle of 45 degrees to the bracings. Bracings at each level were first welded to the four half sleeves, then the lower leg of the post was inserted and welded to the half sleeves. After that the upper length of leg was inserted and welded, and finally the outer halves of the sleeves were fixed. To provide rigidity, small stiffening gussets were included. General construction and joint detail are shown in Fig. 3.

With regard to general building construction, interesting examples are provided by the Post Office Savings Bank Building, Warsaw. Fig. 4 gives the general outline of the Dome, the framework being about 40′-0″ diameter, consisting of a circular base ring of tee section, with rafters and purlins of 2″ diameter tubes. The apex joint where all rafters meet is made with a tubular ring to which the rafters are welded. At the base ring the rafters are slotted to pass over the leg of the tee, and are then welded to the leg and the inside of the table.

Roof trusses in this building, of about 47′-0″ span, are built up with 2″ diameter tubular web members, the rafters and main ties being of tee-section, and the joints being similar to those in the Dome.

In the Austrian Tyrol there is a footbridge of about 46′-0″ span, built entirely in tubular construction. The trusses are of Warren girder type with verticals. Main members are of 3″ diameter tubes and floor bracings of $2\frac{1}{2}$″ diameter tubes. The top chords of the trusses also serve as handrails. To simplify fabrication sleeves were provided to the truss panel points. The weight of this bridge was only about half that of normal rivetted construction, and the tubular design was definitely more economical.

In view of the obvious possibilities of this medium of construction, a considerable amount of research work has been carried out in the United States, covering factory buildings of standard

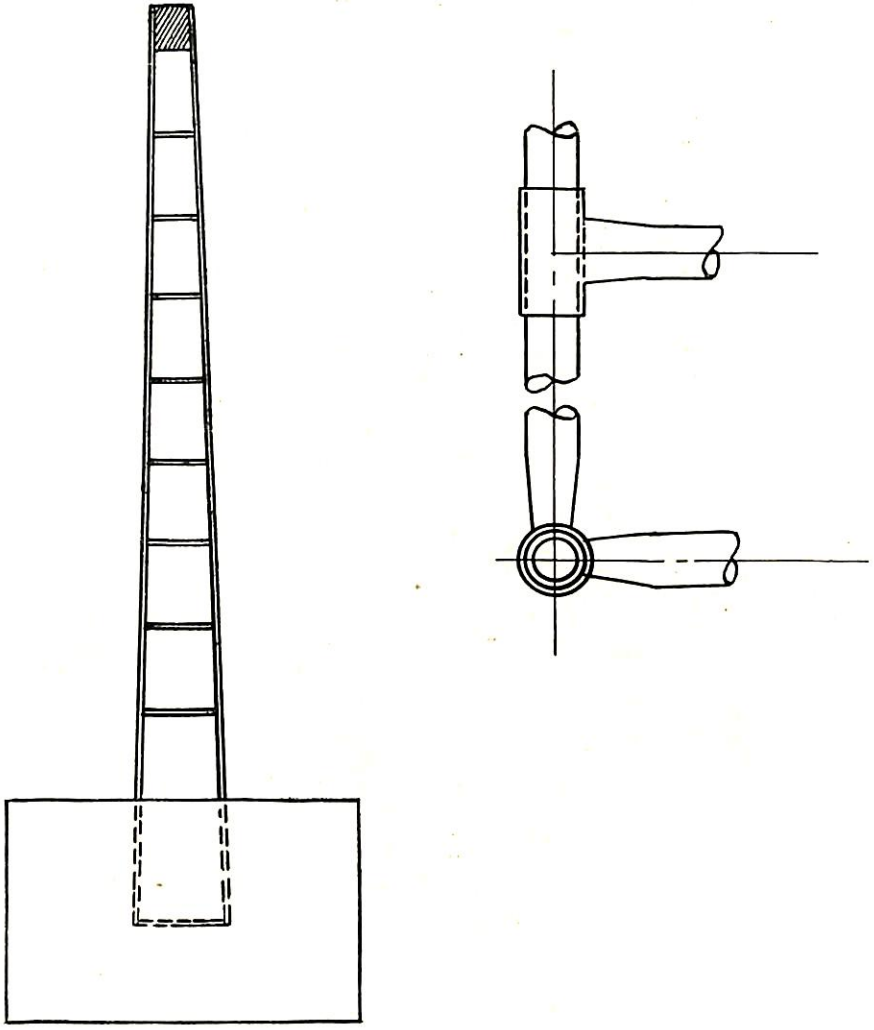


Fig. 3—Transmission Masts.

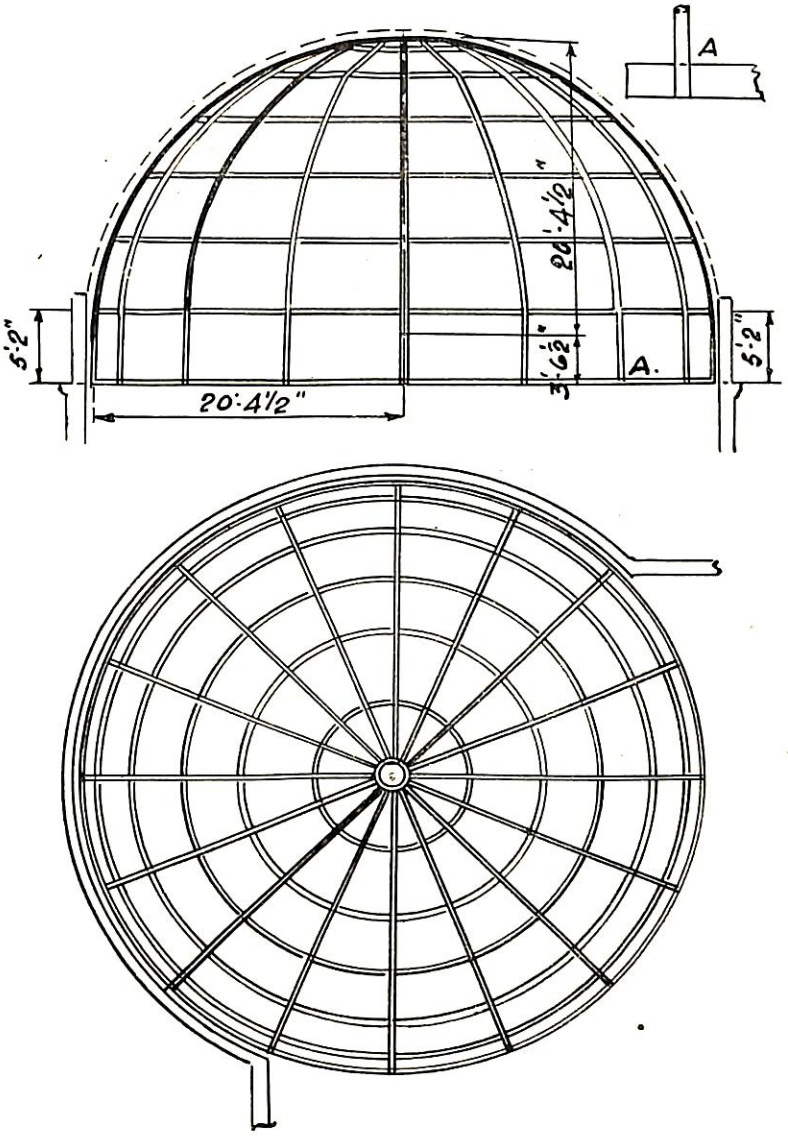


Fig. 4—Dome to P.O.S.B. Building, Warsaw.

types, airplane hangers, bridgework, and elevated tanks. A number of elevated tanks have been built with tubular supports, some of them of striking and novel appearance, as befits a land not too wedded to conservative ideas. A fair amount of general constructional work has also been carried out on the same principle.

In a few cases opportunity has been taken to incorporate old piping, as, for example, the Sublet Bridge, Wyoming, a service bridge nearly 400′-0″ long for coal haulage, consisting of a series of short spans built up of old 10″ diameter steel pipes and scrap rails.

The largest factory buildings constructed almost entirely of tubular sections, with which the author is familar, is Messrs. Stewarts and Lloyds warehouse at Sydney, Australia, consisting of three bays, each 52′-0″ span, two bays being 588′-0″ long and the third bay 564′-0″ long. Apart from the roof beams and crane girders, the framework is entirely of tubes. Stanchions are of normal build, with tubes instead of joists for the roof and crane legs. Trusses are of Fink shape with tubular members, and purlins, side rails, end rails, bracing and framing generally are all of tubes.

General Considerations.

It is necessary first to consider the general suitability of tubular sections for the various kinds of stress met with in design, and the relative merits of tubes compared with rolled sections in resisting such stresses.

Compressive Stresses.

The allowable stress in compression members depends upon the ratio of slenderness—the higher this ratio, the lower is the working stress. For any given sectional area it is therefore desirable to choose a section with the largest possible radius of gyration, which in turn is based upon the moment of inertia.

All normal structural sections, such as joists, channels and angles, have maximum and minimum values for the moment of inertia, the difference between these values being in many cases considerable. As stresses are based on minimum values of the moment of inertia, designers are always faced with the problem of finding ways and means of securing a working stress sufficiently high to avoid undue waste of material. A common device is to reduce the design length of the member about the weaker axis by introducing additional bracing, which, with a more favourable strut section, could be omitted without detriment to the strength of the structure.

No difficulty of this kind is experienced with tubular sections, which so far as compressive stresses are concerned provide ideal sections. The material is distributed uniformly and at the farthest possible distance from the centre of gravity, thus giving the same moment of inertia, the same radius of gyration, and the same

strength about all axes. It is, in fact, impossible to conceive of a more suitable section for compressive loads, so far as design is concerned.

This obviously gives tubes a decisive advantage over rolled sections of all kinds, with their stronger and weaker axes. An equal or greater radius of gyration can be obtained for a smaller sectional area, thus effecting a considerable saving in weight on the whole structure, which is further increased by the elimination of unnecessary bracings.

Tubular sections, by reason of their shape, are also much stiffer against local buckling than rolled sections, and may therefore be supplied with thinner material. This consideration offers substantial advantages in cases where minimum thicknesses of metal are specified for rolled sections, as to meet this requirement a rolled section often has to have far more metal than is strictly necessary for strength. Possible gains in this direction are to some extent limited by the provisions of British Standard Specifications regarding maximum permissible values for working stresses and for ratios of slenderness. B.S.S. 153 for girder bridges fixes a maximum ratio of slenderness of 120, corresponding to a working stress of 4·9 tons, and also a maximum working stress of 7.65 tons, corresponding to a ratio of slenderness of 40. On this basis the maximum saving in weight when using tubular sections for bridgework—so far as permissible stresses are concerned—will be about 60 per cent. This specification was, of course, compiled with rolled sections in mind, and it is at least arguable that greater flexibility could be permitted when using tubular sections. In any case a saving in weight of 60 per cent. is substantial, and goes far to neutralise the higher price per ton of tubular sections.

With regard to building construction generally, a much higher value for the ratio of slenderness is permitted by B.S.S. 449, and the saving in weight here will therefore be much greater.

Tension.

The required sectional area of a member in tension is dependent upon the permissible working stress and any loss of sectional area due to holes, so that any tension member with welded connections will be stronger than the same member with rivetted connections. So far as welded work is concerned, tubes offer no advantage over other sections as tension members, except in cases where specifications state that only a percentage of the area of a rolled section may be assumed to be effective in resisting load. This latter provision applies to angles, and is based on the assumption that the outstanding leg will not take so much load as the rivetted leg.

Where, however, minimum sections are specified, or where for the sake of symmetry a double angle must be included, tubes will invariably work out lighter.

So far as tension members are concerned, it may therefore be stated that tubes will tend to work out lighter than rolled sections, but the saving will not be so large as in the case of compression members.

Bending.

For girder sections proper, carrying fairly heavy loads, tubular sections are obviously unsuitable, but for such members as sheeting rails, purlins, etc., subject to small dead and wind loads, tubes will be much stronger than angles of the same weight per foot, owing to their uniform moment of inertia about all axes. Also, for members subject to direct compression, together with a small bending moment, as for instance roof truss rafters with the purlins located away from the panel points, tubes offer definite advantages.

Generally, tubes merit consideration for beam sections when the span and the bending moment are small, or when a compressive stress has to be withstood in addition to a small bending moment.

Torsion.

Torsional stresses, whether considered or not, occur to a greater or lesser extent in most structures. As they are usually small compared with other stresses they are often ignored completely, or assumed to be covered by the margin of strength in the members. It is well known, however, that rolled sections, by reason of their shape, are very inefficient in this respect, while tubular sections are ideal for the purpose, so that whether or not torsional stresses are taken into consideration the designer is at least assured that in adopting a tubular section he has made the best possible provision against these stresses.

Other Considerations.

Apart from purely statical considerations, which, taken overall, are favourable to tubular sections ; other points require attention which vitally affect the cost of a structure.

Tubular structures will be lighter in weight than others, thus effecting a considerable saving in steel, and economy in materials is always desirable in itself.

Further, the reduced weight means a reduction in the dead load of the structure, and this in turn will again reduce the sections of the principal members whatever the nature of the stresses.

In addition, a plain tubular section may replace a built-up rivetted section such as a compound stanchion, this saving a considerable amount of labour. (See Design 3 later).

Wind loads will also be less on a tubular structure, and in such cases as towers where wind loads are a governing factor in design, this point will be particularly important.

Cost of painting and general maintenance will also be reduced, partly due to the decreased weight, and partly owing to the fact that curved surfaces with no sharp corners or narrow spaces awkward of access facilitate the work of cleaning and painting.

Transport and erection costs, which are usually based on tonnage rates will automatically be reduced, and further savings may be effected by using the tubes for dual purposes. It is comparatively simple to use members for water or service conduits, and for handrailing, thus reducing the costs of ancillary services.

It is hardly possible to reduce all these savings to a percentage, but it will be clear that a considerable saving in many directions can be effected, and the point to be decided in every individual case is whether all these savings taken together will cancel out the higher price per ton of tubular sections.

Further points to which attention must be given are internal corrosion and aesthetics. With regard to corrosion, a tube is no better and no worse than any other totally enclosed member. It has always been assumed that so long as the enclosed air has no contact with the outside air—thus setting up air currents—there is no likelihood of deterioration of the metal. As the ends of tubular members are closed in, or hermetically sealed, there would appear to be little danger of corrosion. In any case tubes can be given an internal protective coating without difficulty, and as a precautionary measure it seems desirable to specify a coating of some kind. Linseed oil is very suitable for this purpose, or in special cases the tubes can, of course, be galvanised, but as the galvanising is burnt off in making welded joints, it is necessary to replace it with a protective coat of suitable paint.

Aesthetics has always been a doubtful point in structural design, and has never received the attention it deserves, as engineers usually have somewhat hazy ideas on the subject. It can be stated, however, that smooth clean construction lines, emphasising the essential nature of the design layout, give a better appearance than designs embracing a large amount of bracing and subsidiary members, which tend to give a confused spider-web appearance to a structure. From such information as is available on this subject it would appear that the smooth curved surfaces of tubes will look better than the irregular broken shapes of rolled sections, especially as with tubular structures fewer members will be required and the main outline of the job will stand out more clearly. This is especially true of towers, where aesthetics is a primary consideration, and explains to some extent the attention paid to tubular construction for this class of work.

General Procedure and Properties of Sections.

It is now possible to consider the general procedure for design of members and connections. Generally, this will, of course, follow

normal structural practice, modified only by considerations and problems raised by the shape of the members and the physical properties of the material.

Basic design expressions for properties of tubes are as follows :—

Let D = outside diameter of tube.
d = inside diameter of tube.

Then A (area of section) $= \frac{22}{7}\left(\frac{D^2 - d^2}{4}\right)$

I (moment of inertia) $= \frac{22}{7}\left(\frac{D^4 - d^4}{64}\right)$

I_P (polar moment of inertia) $= 2 \times I$

Z (modulus of section) $= \frac{2 \times I}{D} = \frac{22}{7}\left(\frac{D^4 - d^4}{32\,D}\right)$

r (radius of gyration) $= \sqrt{\frac{I}{A}} = \sqrt{\frac{D^2 + d^2}{16}}$

Tables 1 and 2 give properties of hollow column sections of standard thickness, and Table 3 gives similar properties of thin hollow column sections.

Tables 4 and 5 give properties of light weight and heavy weight tubes, made to B.S.S. 789A, while Tables 6 and 7 give similar properties for Class B and Class C tubes made to B.S.S. 1387, which supersedes B.S.S. 789A.

In all seven tables, the outstanding feature is the value of the radius of gyration, and comparison of these values with those for rolled sections of similar weights per foot will demonstrate the superiority of tubes as compression members, and also give some indication of the possibility of omitting unstressed stiffening members necessary in normal structural practice.

Tables 8 and 9 give loads which can be carried by light weight and heavy weight tubes for various stresses, while Tables 10 and 11 give corresponding values for Class B and Class C tubes.

It might be noted for reference purposes that Class B tubes to B.S.S. 1387 are one gauge heavier than light weight tubes of the same bore, and one gauge lighter than heavy weight tubes, while Class C tubes are one gauge heavier than heavy weight tubes.

Tubular structures will be designed to meet the requirements of appropriate specifications, although it should be noted that these specifications were compiled with rolled sections in mind, and also primarily for rivetted or bolted structures. With the two-fold variation involved in using tubes with welded connections, certain adjustments to specification are justifiable and desirable, and these should be agreed before the design is commenced.

First important point requiring attention is the value of the permissible stresses. Mild steel rolled sections to B.S.S. 15 have an ultimate tensile stress of 28-33 tons per square inch, and working stresses corresponding to this ultimate value are given in B.S.S. 449 for Use of Steel in Buildings, and in B.S.S. 153 for girder bridges. Stresses in B.S.S. 449 were temporarily increased as a war-time economy by the War Emergency Revision, and since 1945 various Codes of Practice have been issued with a view to stabilising stresses at a higher level. Recently a new (1948) edition of B.S.S. 449 has been published, based on the higher stresses in the above codes.

No increases of any kind have been made to allowable stress in B.S.S. 153, and none appear to be likely in the foreseeable future.

B.S.S. 789A and 1387 for Steel Tubes both specify an ultimate tensile stress of 22-30 tons per square inch, which gives a much lower limit than for mild steel sections. These specifications, however, were intended to cover the use of tubes for normal tubular constructions rather than for the purposes referred to in this pamphlet. There is no reason whatever why tubes should not be manufactured to any tensile desired. Both on the Continent and in this country they have in fact been made in large quantities for many years, in both mild steel and high tensile steel qualities, and to ensure the full economy of tubular construction, it is essential that the material should comply with the requisite structural specifications.

In this pamphlet the procedure adopted is to design to B.S.S. 153 in the case of bridgework, and to B.S.S. 449 (1948 edition) for general building work. In any case it is a simple matter, if it should be necessary, to modify the procedure to suit future tubular specifications.

With regard to permissible stresses in struts and columns given in B.S.S. 153 and 449, further comments appear to be called for. As is well known, most strut and column formulae are based on experimental data which take into account the shape, length, type of loading, and degree of end fixity, and it is difficult to see on what other basis it is possible to proceed.

So far as bridgework is concerned, there seems to be no justification for departing from the formulae given in B.S.S. 153, as these assume the designer will seek a section as nearly as possible to the ideal strut, that is to say, as nearly as possible to the condition provided by a tube.

With building work, however, there is room for some divergence of opinion. In the first place, the formula given in B.S.S. 449, although stated to be applicable to both columns and struts, is primarily a "column" formula, and although a column may be a strut, it is not exactly the same kind of strut as occurs in a braced

frame. Also, this column formula is devised to take into account the shape of the normal column section and irregularities due to uneven rolling. There is, in fact, considerable doubt as to how far the formula in B.S.S. 449 should be employed, and in the case of braced frame struts there appears to be no valid reason for designing these on any other basis than that given in B.S.S. 153.

From the design point of view, tubular sections are much more satisfactory than rolled sections, both as building columns and as braced frame members. So far as short columns and struts are concerned, it would appear from such experimental data as exists that the Johnson Formula for Short Struts gives more satisfactory results for tubular work than the formula in B.S.S. 449. This point will be made clearer by an examination of the values for permissible stresses given in Tables 12 to 15.

Table 12 gives permissible stresses in mild steel struts, with pin-jointed ends for axial loads, in accordance with B.S.S. 153 for girder bridges, and within the range of slenderness ratios usually met with in tubular construction.

Table 13 gives corresponding values with rivetted (or partially fixed) ends, and in both tables the maximum stress allowed is 7·65 tons.

Table 14 gives permissible stresses in mild steel columns for axial loads, in accordance with B.S.S. 449, the values given being for a fixing factor of unity, which corresponds to pin-jointed ends, so that this table is for a condition comparable with that of Table 12. In this case, however, the maximum stress allowed is 9 tons. Values given are based on a yield stress of 18 tons per square inch and a factor of safety of 2, so that for a very short strut of ideal shape the working stress, in accordance with this formula, becomes $18/2 = 9$ tons, which is higher than the maximum allowable stress in Tables 12 and 13.

Table 15 gives the permissible stresses in mild steel struts for axial loads in accordance with the Johnson formula and within the range of slenderness ratios for which this formula is suitable. The formula is empirical, and values have to be inserted for f_c and g. The usual value for g in the case of mild steel is 0·000057, and f_c (the permissible stress in a very short strut) has been fixed at 9, to give values comparable with those in Table 14.

Comparing all these Tables, it will be clear that Table 13 should give higher values than Tables 12 and 14, as the condition is for partially fixed ends. Tables 12 and 14 give very similar values, except where the ratio of slenderness is small, while Table 15 (Johnson's formula) does not differ very greatly from Table 13.

Before finally deciding which formula to adopt in any particular case, attention should be given to the degree of rigidity in the end connection. Tables 12 and 13 naturally show a marked difference between pin-jointed and rivetted ends, and the differences increase

with every increase in the slenderness ratio. As, however, tubular sections usually have a low slenderness ratio, this point is not so important in practice with tubes as with rolled sections. Also, as the actual stress will be less than the permissible, it will often be found that with tubular struts this particular problem will not cause very much difficulty, certainly less than with other constructions. Welded joints are obviously more rigid than pin joints, but not so rigid as the type of rivetted joint visualised in B.S.S. 153.

Summarising all the relevant factors mentioned above, the writer's recommendations in cases where no specification is quoted are as follows :—

(1) For columns with a ratio of slenderness exceeding 80, the formula given in B.S.S. 449 should be used. (Table 14).

(2) For columns with a ratio of slenderness of less than 80, the Johnson formula should be used. (Table 15).

(3) For braced frame struts generally, whether for bridgework or building work, a value should be adopted between those given in Tables 12 and 13, depending upon the degree of end fixity provided by the end connections. Generally a value midway between those in Tables 12 and 13 will be satisfactory.

Permission to proceed on the above lines must, of course, be obtained.

Tables 16, 17 and 18 give safe concentric loads on hollow columns from 18″ to 6″ diameter, in accordance with the formula in B.S.S. 449. Values to the right of the heavy line have ratios of slenderness of less than 80, and corresponding values for safe loads on such columns in accordance with Johnson's formula, are given in Tables 19, 20 and 21.

Comparing these two sets of tables, it will be seen that for a ratio of slenderness of 95 the difference between the two formulae is small, but for shorter columns, Johnson's formula gives gradually increased values for the safe load.

Tables 22 to 25 give safe loads on light weight and heavy weight tubes, as struts, in accordance with the formulae in B.S.S. 153 for girder. bridges, while Tables 26 to 29 give similar values for Class B and Class C tubes.

For certain types of connections a decision will be necessary with regard to permissible stresses in the fillet welds. With rolled sections there is a clear distinction between side fillets and end fillets, and all such welds are also reasonably constant in cross section. In the case of tubular connections it is not always possible to state with exactness what proportion of a weld may be regarded as a side fillet and what proportion is an end fillet. This is due to the circular profile.

Also in certain joints the cross section of the weld will vary from an acute angle to an obtruse angle. L.C.C. regulations, issued in December, 1937, specify a permissible stress of 6 tons per square inch for end fillets and 5 tons per square inch for side fillets, so that with all such connections of the type mentioned above it seems desirable to work to a stress of 5 tons, based on the minimum cross section. This procedure will at least ensure that adequate strength has been provided.

Attention should also be given to the fact that in many tubular joints the intersection point will occur at what may be described as a metal-empty space, thus setting up a certain amount of bending moment in the joint which should be taken into consideration when the loads in the members are substantial. The same thing occurs of course when using unsymmetrical rolled sections such as angles or channels, so that the problem is not peculiar to tubes, and, in fact, may not be so important in tubular construction as in other forms. This point is quite commonly ignored in structural design, but while the bending moment may be small in particular cases and probably covered by the surplus strength provided, it deserves more attention than it commonly receives.

Table 30 gives particulars of suitable pipe flanges for structural work.

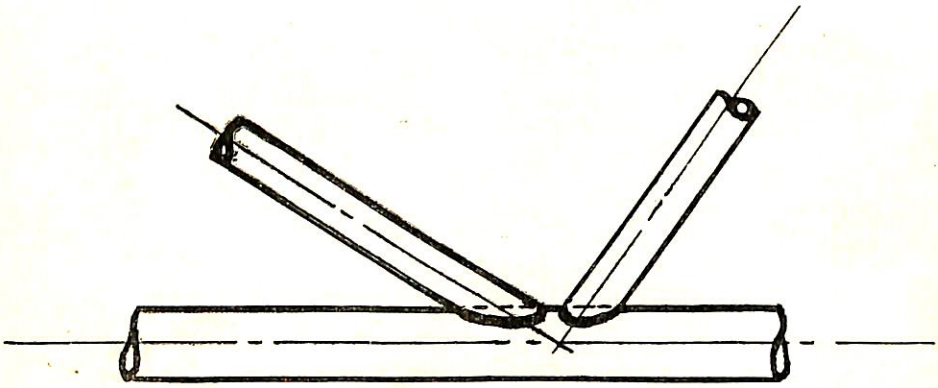


Fig. 5.

Types of Connections.

Typical details of standard connections for tubular construction are shown in Figs. 5 to 26, and also in Fig. 2.

In Fig. 5 the subsidiary members are welded direct to the main member. Connections of this type are usually reserved for members carrying small loads, as, for instance, the subsidiary members of a roof truss, as apart from some doubt regarding the actual strength of the joint fairly high local stresses may be set up when the metal

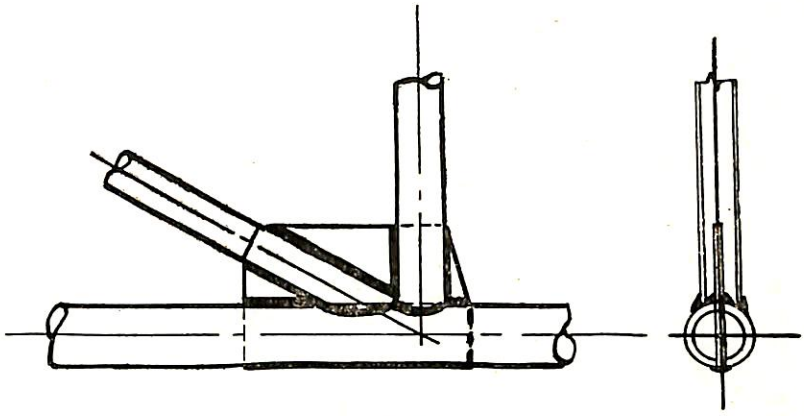


Fig. 6.

is thin. This defect may be remedied by inserting short sleeves at the joint in the main member, or by introducing small stiffening gussets.

Fig. 6 shows a much stronger detail, although, of course, more expensive. In this case all the tubes are slotted and welded to the gusset. Joints of this type are very satisfactory, and also meet the criticism of the intersection point occurring at a metal empty space.

Fig. 7 is an alternative to Fig. 6, and shows the gusset plate welded to one side only of the main member. Naturally this is cheaper, and may be quite satisfactory when loads are small, but the construction is obviously not so sound as that shown in Fig. 6.

Fig, 8 gives a further alternative, also illustrated in Fig. 2, in which the bracing members are welded to the gusset only. Joints of this type are satisfactory for small loads, but again are not so good as the procedure in Fig. 6. In order to seal the tubes against

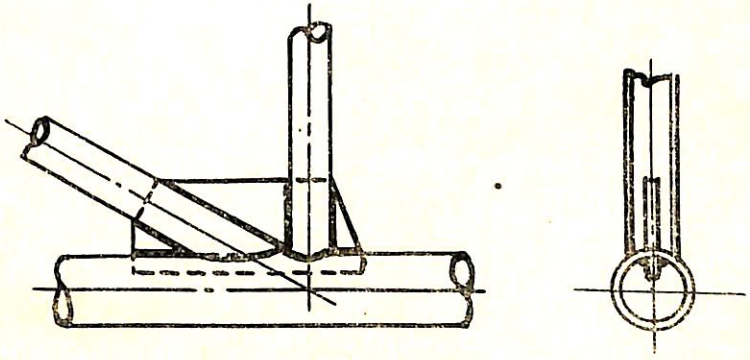


Fig. 7.

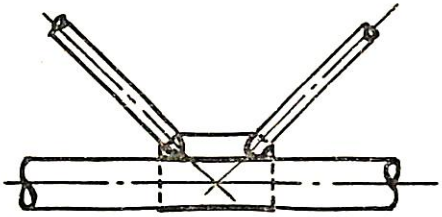


Fig. 8.

internal corrosion, the forked ends must be bent down on to the gussets and welded all round.

Structures may also be built up with a combination of tubes and rolled sections, the section most convenient for this purpose being a tee. For example, roof trusses or light lattice girders may be constructed with chord members of tee-section, and web members of tubes. Joints may then be made, as shown in Fig. 4, by slotting the ends of the tubes to pass over the stalk of the tee and welding the tubes to the stalk and to the inside of the table.

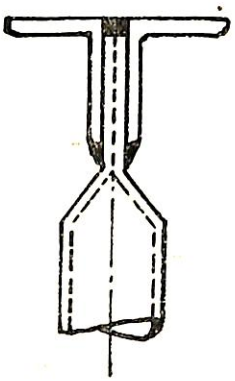


Fig. 9.

Fig. 9 shows tubular web members used in conjunction with double angle chords. Welds at the angle toes connect the angles to the wall of the tube, the other weld joining the tube solidly to the angles and also tying together the corners of the angles. No preparation of material is needed to make these joints, which is obviously an advantage.

Figs. 10, 11 and 12 show further examples of connections for tubes with flattened ends, suitable for either roof truss or lattice girder construction.

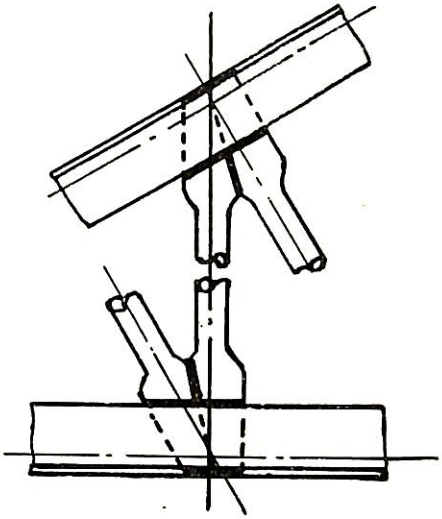


Fig. 10.

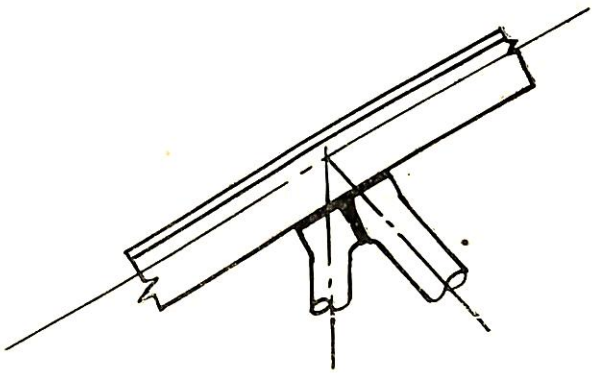


Fig. 11.

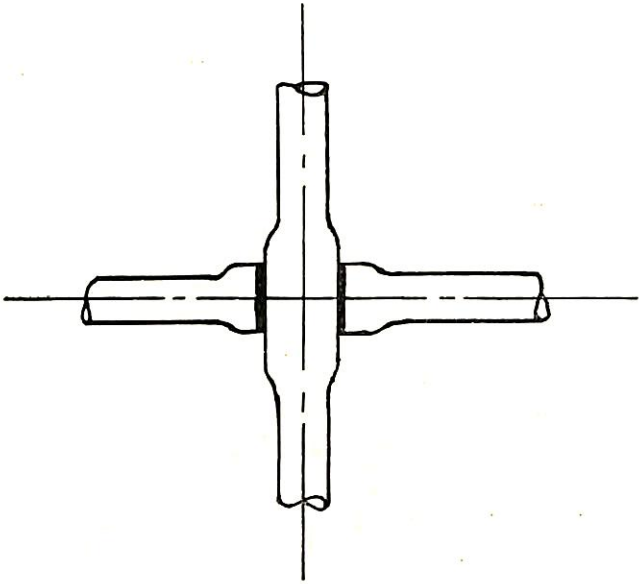


Fig. 12.

Figs. 13 and 14 show typical procedures for lattice girder work. For the example shown in Fig. 14, chords may be cut from rolled joists in the manner shown in Fig. 15, thus saving the gusset welding but involving a certain waste of material. The procedure shown in Figs, 15 and 16 for using split joists can, in fact, be used with advantage in suitable cases for both welded and rivetted work, mainly for continuous members, subject to increment of load. Roof truss rafters and lattice girder chords are typical examples, and the procedure seems to justify more attention than it has received to date. Truss rafters are invariably made of the same section throughout, although the load varies in each panel. Split joists could thus be used, giving maximum area at the shoe and minimum area at the apex. Similarly, with lattice girder chords, maximum area is required in the centre and minimum area at the ends. There appears to be no valid reason why the procedure should not be adopted more regularly, as it is economical of material. In conjunction with tubular construction it could certainly be introduced with advantage. A common objection to the method is that the designer has no time to work out the varying values of the moment of inertia, but for all practical purposes it is sufficient to calculate on the assumption of a rectangular flange, which does not present very great difficulty.

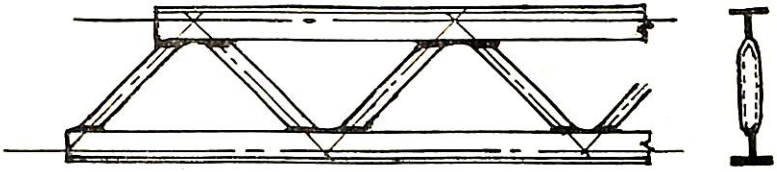


Fig. 13.

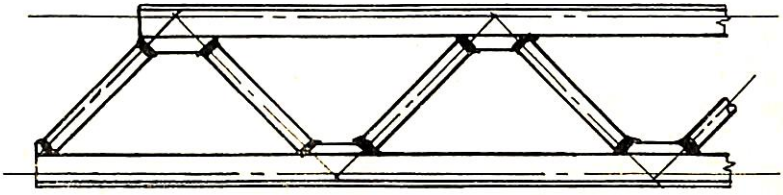


Fig. 14.

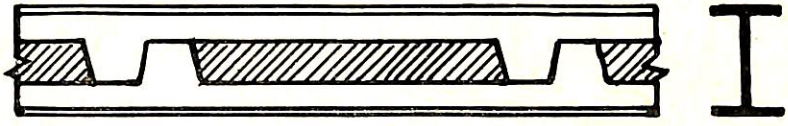


Fig. 15.

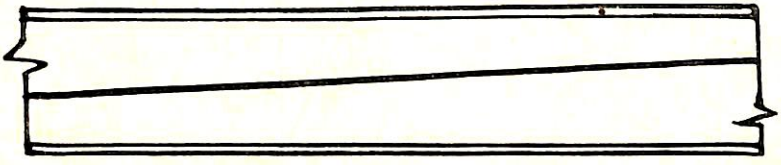


Fig. 16.

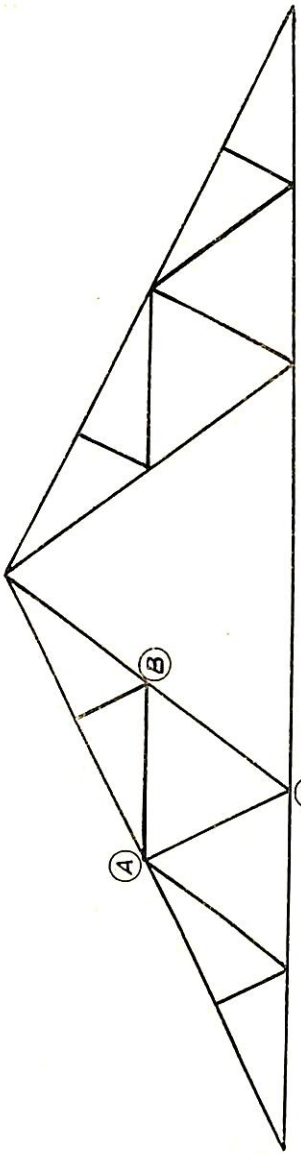


Fig. 17—Roof Truss Joints. Details for Welded Steel Tube Fink Truss.

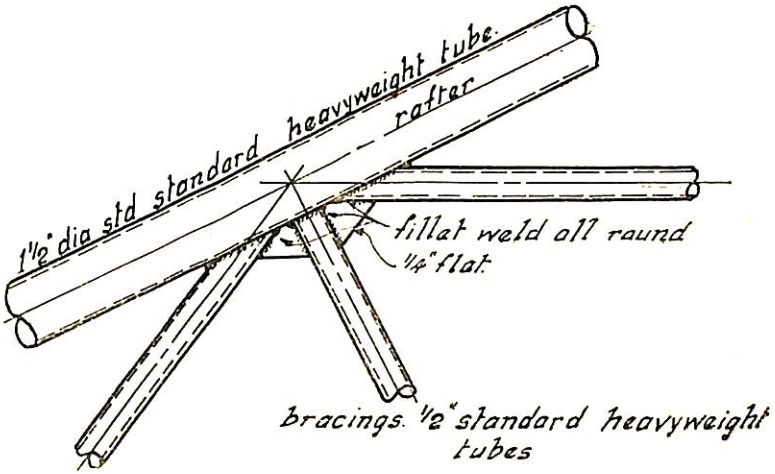


Fig. 18—Roof Truss Joints. Detail at "A" Rafter.

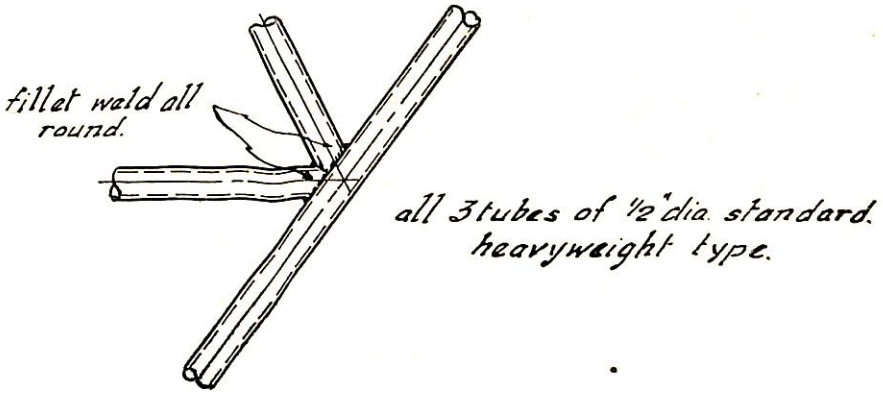


Fig. 19—Roof Truss Joints. Detail at "B" Bracings.

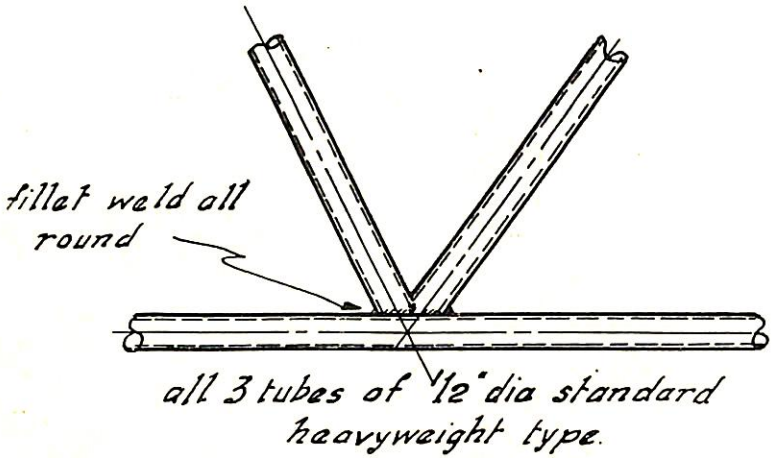


Fig. 20—Roof Truss Joints. Detail ot "C" Bottom Chord.

Typical examples of joint details for a 45′ 0″ span roof truss of standard shape are given in Figs. 17 to 23, and will be self explanatory after previous notes and details.

Figs. 24, 25 and 26 illustrate standard base, cap, and floor connection details for hollow circular columns. This type of construction is particularly suitable and economical for internal stanchions in multi-storey buildings, where incoming beams usually connect centrally to the columns. As for such cases plain circular columns will replace plated sections, the saving in workmanship—apart from saving in weight—will be considerable.

For exterior columns in buildings of this type, where wall beams are usually eccentric to suit architectural requirements, circular columns are not always suitable, and rolled sections may give a more satisfactory construction.

Fig. 27 shows a spherical joint which has been used successfully in America, even in cases where fairly large loads have to be transmitted. A cast steel sphere with a reinforcing rib is introduced at all intersection points in a frame, with the main member passing right through and the bracings welded to the sphere.

Bracing members have shaped ends to suit the profile of the sphere, and all joints are sealed to prevent internal corrosion.

For large frames which have to be assembled at site, the spheres are shop-welded to the main members, with bracings site welded. Alternatively, in order to minimise the amount of site welding, bracing members which are in compression—and also those in tension with small loads—can be flanged near the joint, the short

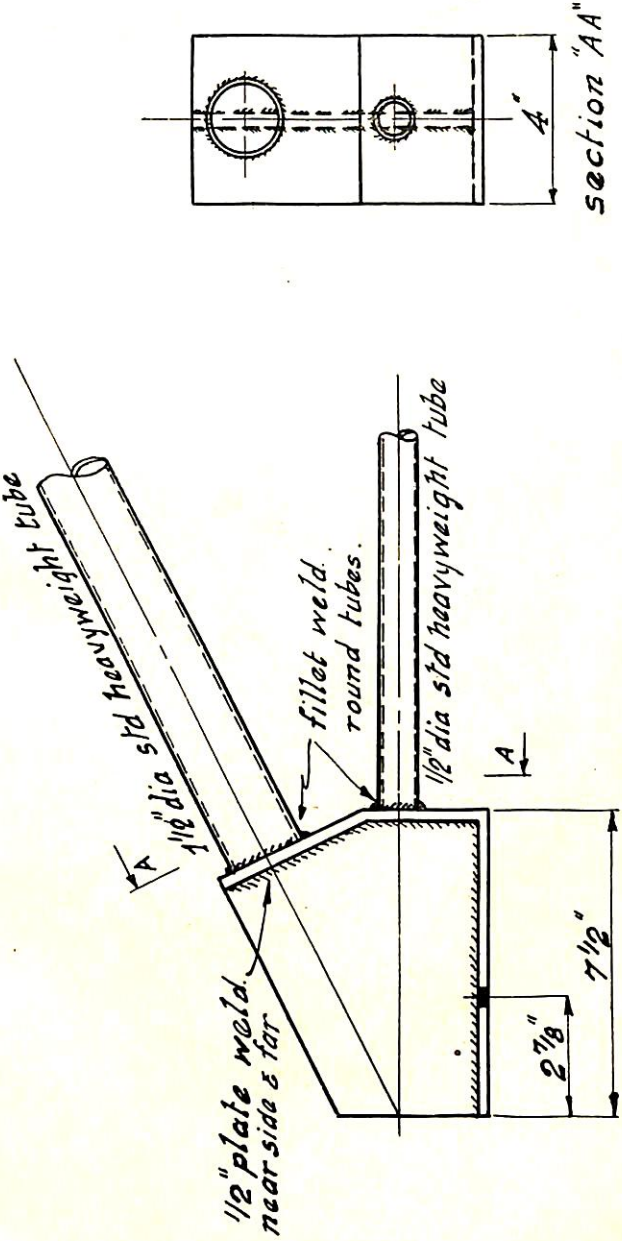


Fig. 21—Roof Truss Joints. Detail of Shoe.

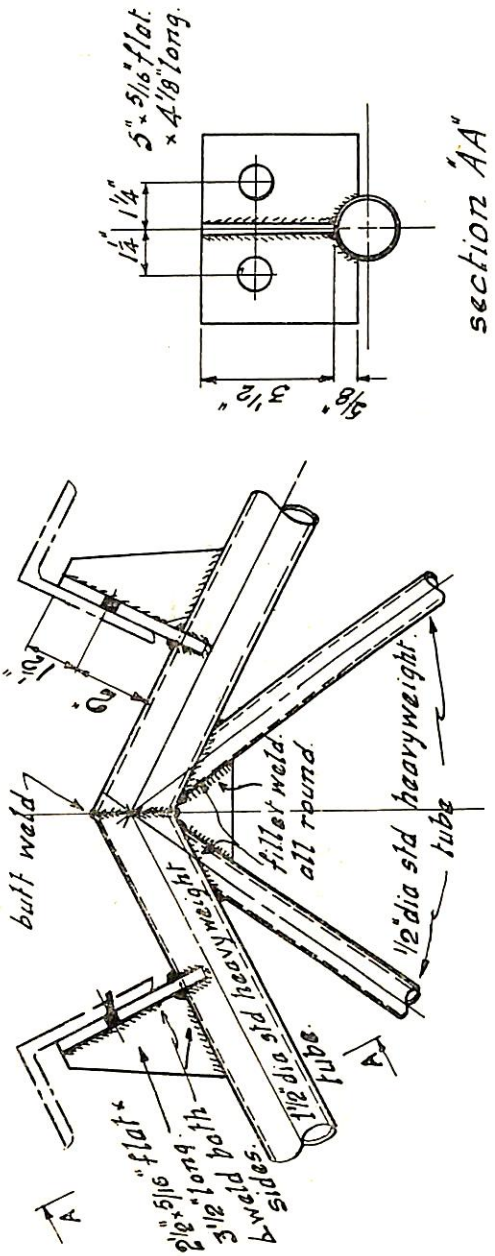


Fig. 22—Roof Truss Joints. Detail at Apex and Purlins.

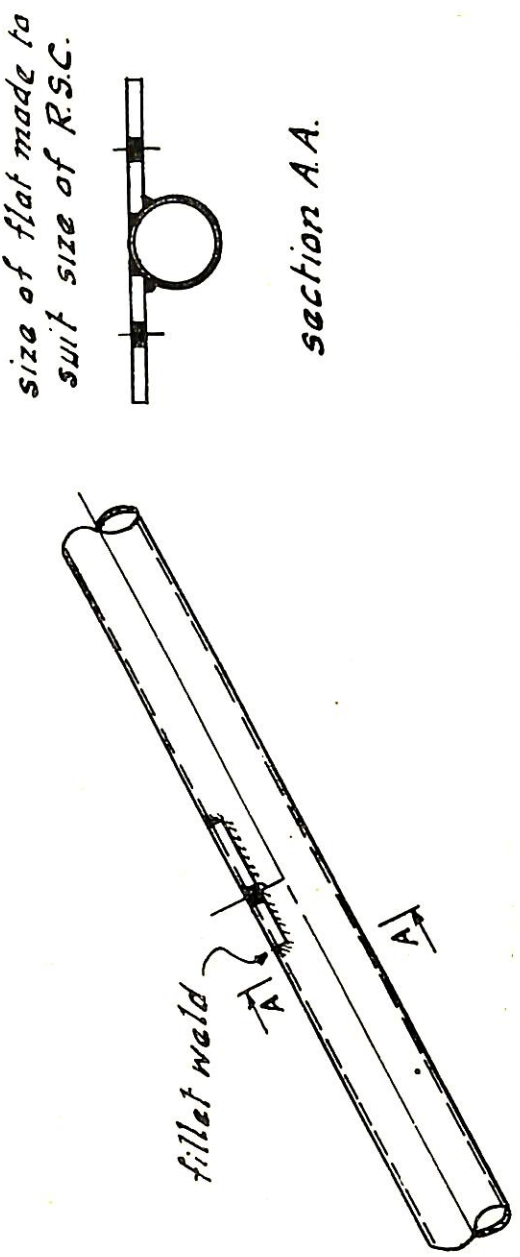


Fig. 23—Roof Truss Joints. Alternative detail for R.S.C. Purlins.

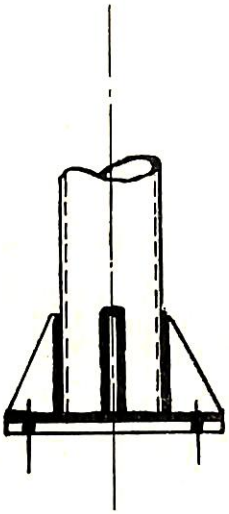


Fig. 24

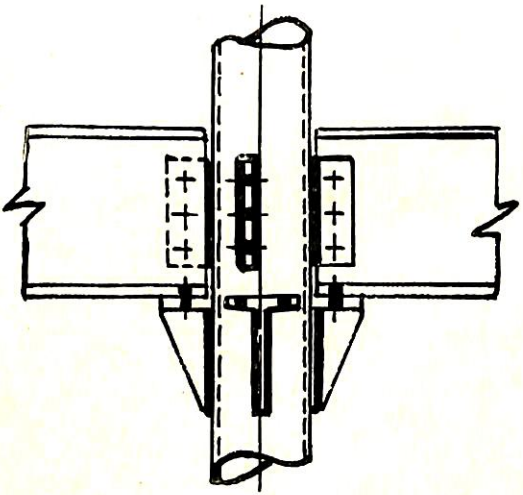


Fig. 25.

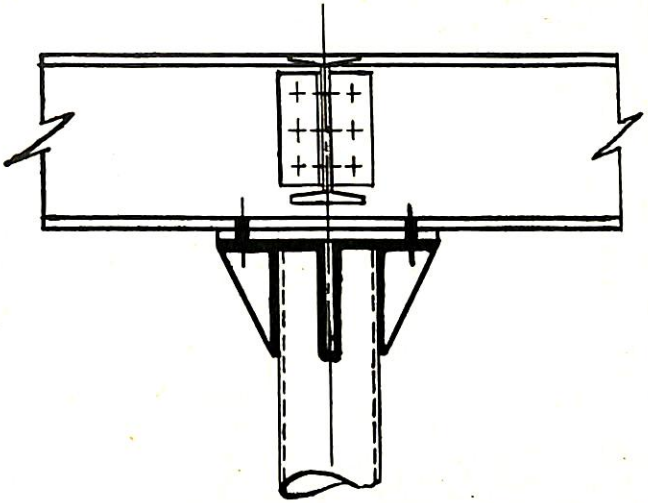


Fig. 26.

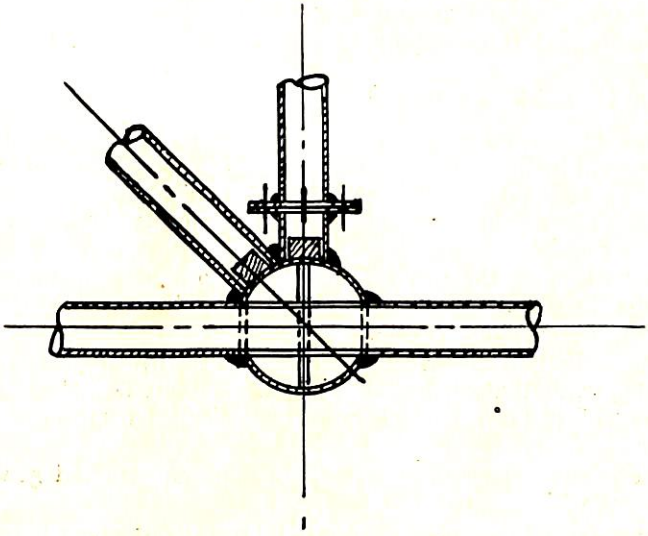


Fig. 27.

length being shop welded to the sphere and the site connection being made with bolts, as shown in the diagram.

An advantage of this type of joint is that the cross section of the weld is constant, thus simplifying calculations and welding operations.

If desirable, the spheres can be provided with locating collars, as shown in the sketch, in order to facilitate erection.

There appears to be some doubt as yet with regard to the correct size of sphere to be used in any particular case, as the procedure is still in little more than the experimental stage, but they should have a diameter of at least one and a half times that of the largest member—usually the main member—and should be large enough to allow each bracing member to be welded independently without any overlapping of welds. The thickness of the sphere should be at least one and a half times that of the thickest tube. This type of joint, though rather novel and seemingly unorthodox in the structural sense, appears to have distinct possibilities, as it gives a satisfactory and straightforward construction. It would be an advantage if some research work could be undertaken into the construction generally, and particularly into the matter of economic sizes.

Comparative Designs.

It is now possible to prepare a few comparative designs, indicating the general procedure for tubular design, and comparing the results with those obtained from the use of rolled sections.

Design 1. 80′-0″ Span Roof Truss.

Figs. 29 and 30 give comparative sections for a rivetted truss, constructed in the normal manner, with angle sections and gussets, and for a welded tubular truss. Both designs are for the same standard dead, wind and suction loads, loading particulars being given in Fig. 28.

In both cases the rafters are subject to a bending moment in addition to direct compression, caused by the purlins occurring away from the panel points.

The weight of the rivetted truss, built with angles, is about 28 cwts., while that of the tubular truss is about 19 cwts., a saving of about 32 per cent., which is fairly typical of results for braced frames generally.

The joints in the tubular truss will be constructed generally, as shown in Figs. 18 to 23.

Design procedure for the rivetted truss is not given here, as it follows normal structural practice. Procedure for the tubular truss is given below.

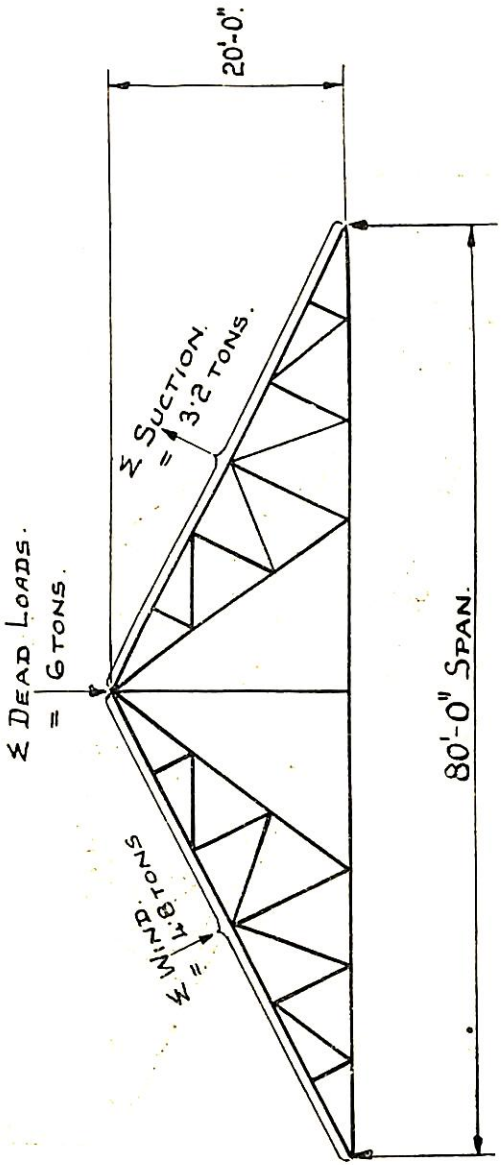


Fig. 28—80 ft.-0 in. Span Roof Truss.

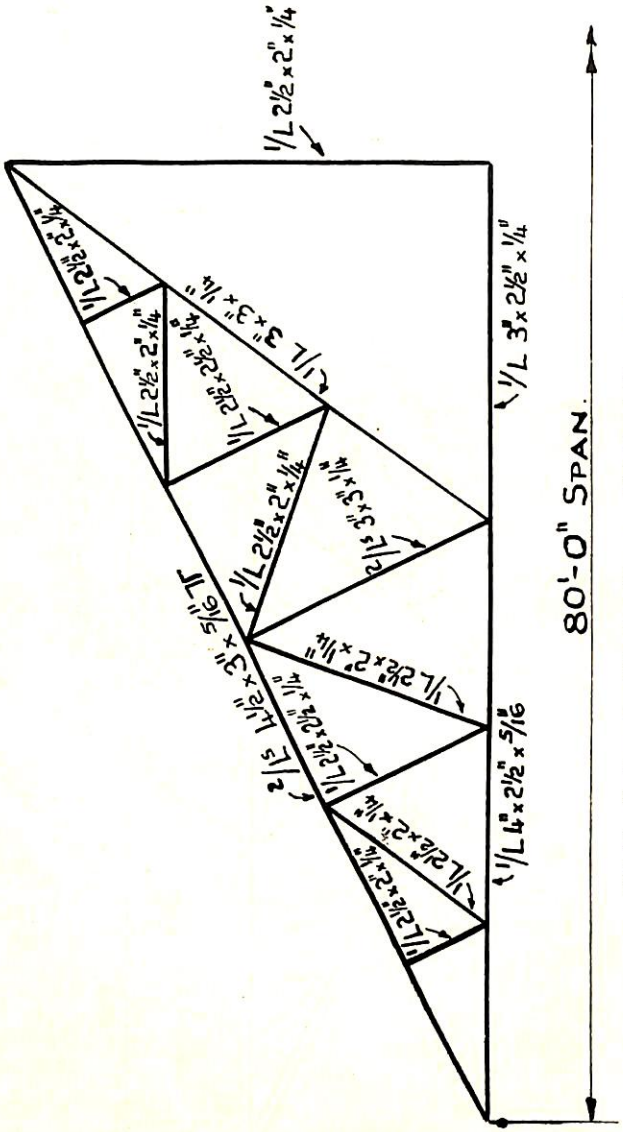


Fig. 29—Sections for Riveted Truss. Weight of Truss: 28 cwts.

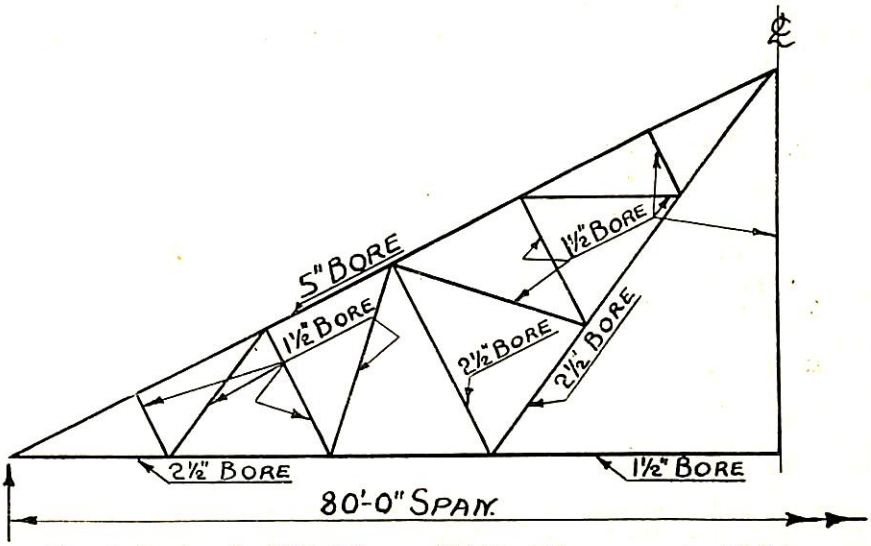


Fig. 30—Sections for Welded Truss. Weight of Truss: 19 cwt. All Tubes are Class "B" Tubes.

Main Rafters.

Maximum direct compression = 12 tons.
B.M. due to purlins off panel points = 16 tons/ins.

Try 5" bore, Class B Tube.

Area of section = 2·94 sq. ins.
Modulus of Section Z = 3·80
Radius of gyration = 1·875 ins.
Panel length = 90 ins.

For fixing factor of 0·8 design length = 90 × 0·8 = 72 ins.

$$L/r = \frac{72}{1{\cdot}875} = 38{\cdot}4 \quad \therefore \; F_a = 7{\cdot}14 + 25\% \text{ for wind} = 8{\cdot}92 \text{ tons per sq. in.}$$

$$\text{Direct stress} \quad f_a = \frac{12}{2{\cdot}94} = 4{\cdot}8 \text{ tons per sq. in.}$$

$$\frac{f_a}{F_a} = \frac{4{\cdot}8}{8{\cdot}92} = 0{\cdot}54$$

$$\text{Bending stress} \quad f_{bc} = \frac{16}{3{\cdot}80} = 4{\cdot}21 \text{ tons per sq. in.}$$

$$\text{Allowable bending stress} \quad f_{bc} = \frac{1000}{L/r} \times K1$$

$$\frac{r_x}{r_{yy}} = 1. \quad \therefore \; K1 = 1{\cdot}5 \text{ (from B.S.S. 449)}$$

$$\therefore \quad F_{bc} = \frac{1000}{38{\cdot}4} \times 1{\cdot}5 = 10 \text{ tons maximum} + 25\% \text{ for wind.}$$

$$= 12{\cdot}5 \text{ tons per sq. in.}$$

$$\therefore \quad \frac{f_{bc}}{F_{bc}} = \frac{4{\cdot}21}{12{\cdot}25} = 0{\cdot}34.$$

$$\frac{f_a}{F_a} + \frac{f_{bc}}{F_{bc}} = 0{\cdot}54 + 0{\cdot}34 = 0{\cdot}88$$

This value is less than unity and the section is therefore satisfactory.

Main Strut.

Load = 3·6 tons compression.
Length = 132 ins.

Try $2\frac{1}{2}$" *Bore, Class B Tube.*

Area of section = 1·56 sq. in.
Radius of gyration, r = 1·01 ins.
For fixing factor of unity L/r = 132.
Allowable stress = 2·83 tons per sq. in.
+ 25% for wind.
= 3·54 tons per sq. in.

Allowable load = 3·54 × 1·56 = 5·5 tons.

Section is therefore satisfactory.

Secondary Struts.

Maximum load = 1·9 tons.
Maximum length = 88 ins.

Try $1\frac{1}{2}$" *Bore, Class B Tube.*

Area of section = 0·876 sq. ins.
Radius of gyration r = 0·623 ins.

For fixing factor of unity $\frac{L}{r} = \frac{88}{0{\cdot}623} = 142$

Allowable stress = 2·52 tons per sq. in. + 25% for wind.
= 3·15 tons per sq. in.

Allowable load = 3·15 × 0·876 = 2·8 tons.

Section is therefore satisfactory.

End Main Tie.

Load = 12 tons tension.

$$\text{Area of section required} = \frac{12}{9+25\%} = \frac{12}{11{\cdot}25} = 1{\cdot}07 \text{ sq. ins.}$$

Use $2\frac{1}{2}$" *Bore, Class B Tube.*

Area of section provided = 1·56 sq. ins.

Apex Tie.

Load = 7 tons tension.

Area of section required = $\frac{7}{11{\cdot}25}$ = 0·63 sq. ins.

Use $1\frac{1}{2}$" *Bore, Class B Tube.*

Area of section provided = 0·876 sq. ins.

Note.—There is a very small amount of compression in this member, due to suction effect, acting as an alternative load to the tension, but the section provided will be ample for this. In cases where the alternative load is considerable, it is necessary to check the member for both tension and compression. (See procedure in Design 4).

All Other Bars.

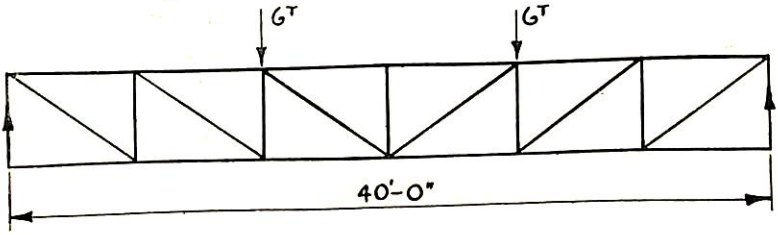
Loads are small, and by inspection these will all be made of $1\frac{1}{2}$" bore, Class B tube, for uniformity.

Notes on Design.

(1) Allowable stresses in compression members are taken from the tables in B.S.S. 449. Values of these stresses for ratios of slenderness up to 120 are also given in Table 14.

(2) B.S.S. 449 specifies that allowable stresses may be increased by 25 per cent. when loads include wind effect. This accounts for the increased values of allowable loads and stresses in the calculations.

(3) The tubular design has been based on the same layout of web members as for the rivetted design, in order to obtain a comparative weight. By introducing a slightly larger size of tube for the main rafters it would, however, be possible to reduce the number of panels in the rafter from six to four without increasing the sections of any other members, thus reducing the total weight



MAX. COMPRESSION IN UPPER CHORD = 32 TONS.
MAX. TENSION IN LOWER CHORD = 32 TONS.
MAX. COMPRESSION IN VERTICALS = 6 TONS.
MAX. TENSION IN DIAGONALS = 10 TONS.

Fig. 31—Layout and Loads for Lattice Girder.

still further and also the cost of manufacture. It may be noted for roof truss design generally when bending occurs in the rafters that it is usually cheaper to reduce the number of panels in the rafter below the practice generally obtaining in English layouts. Incidentally, English practice usually fixes the centres of roof trusses much too close for economy. Economy of weight and workmanship is obtained with a minimum spacing of 15′-0″ and a maximum of about 20′-0″, although the building layout and general requirements usually restrict the designer's choice.

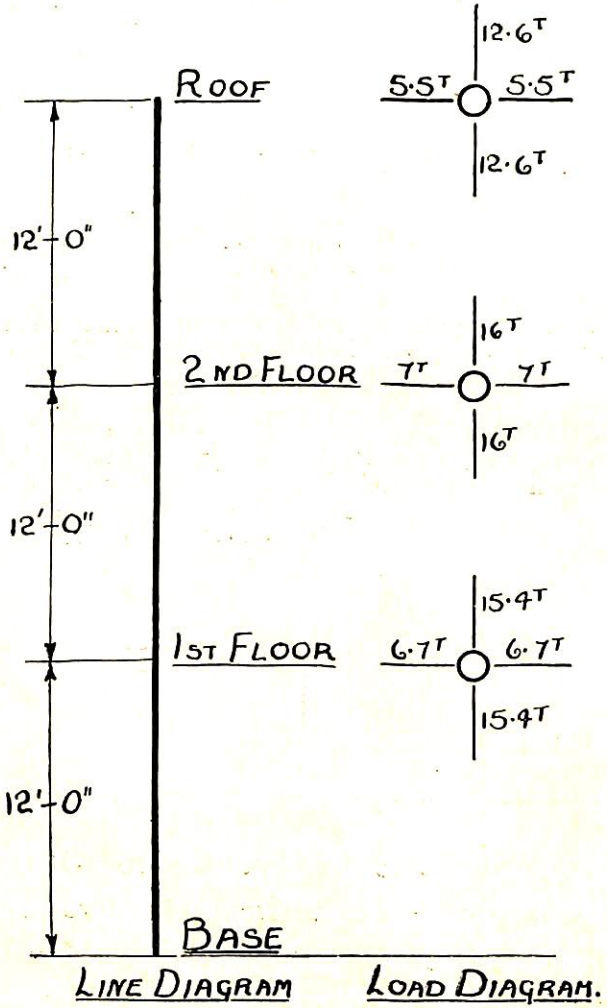


Fig. 32—Stanchion for Office Building.

Design 2—40′-0″ Span Lattice Girder 5′-0″ deep.

A line diagram, with maximum loads in the members, is given in Fig. 31. The upper chords will require to be two angles, 6″ × 3½″ × ⅜″, with 6″ legs turned outwards, and the lower chord will be two angles, 4″ × 3″ × ⅜″, with 4″ legs vertical.

If the web members are also made of angles, two angles, 2½″ × 2″ × ¼″ weighing 7·2 lbs. per foot will be required.

With tubular construction it would be advisable in this case to keep the chord sections as given above, replacing the web members only by tubes.

Proceeding on a similar basis to that adopted in the roof truss design, all the web members can conveniently be made of 2″ bore, Class C tubes, weighing 4·49 lbs. per foot, a saving of about 40 per cent. in the weight of these members.

Connections of the web members to the chords would be made generally, as shown in Fig. 9.

Design 3—Internal Stanchion for Office Building.

Fig. 32 gives a line diagram of the stanchion with loads at roof and floor levels. Following normal design procedure the basis section will be a 14 × 6 × 46 lbs. rolled steel joist with a 12″ × ½″ flat on each flange up to second floor level, the weight per foot below second floor being 89·3 lbs.

For a hollow circular column, procedure is as follows :—

Total load on lower length (including own weight) = 127 tons.

Length from base to centre of first floor connection = 11′-0″ = 132″.

For the condition shown, a fixing factor of 0·85 will be assumed.

Design length = 132 × 0·85 = 112·2″

Try 12″ outside diameter tube ½″ thick at 61·4 lbs. per foot.

Radius of gyration, r = 4·07 ins.

$$\frac{L}{r} = \frac{112{\cdot}2}{4{\cdot}07} = 27{\cdot}5$$

Using Johnson's formula for short struts (Table 15) :

Permissible stress = 8·61 tons per sq. in.

Area of section = 18·1 sq. ins.

Allowable load = 18·1 × 8·61 = 157 tons.

Section is therefore satisfactory.

(*Note.*—If formula B.S.S. 449 is adopted, permissible stress is 7·65 tons, and allowable load 138 tons, so that the section may be assumed to be satisfactory on either basis.

The weight of the circular column is about 70 per cent. of that of the built-up column, but this is not the only saving. The only workmanship on the circular column will be at the connections,

whereas on the built-up column there will be, in addition, the drilling and rivetting for the flange plates. Furthermore, the built-up column will probably be encased in concrete, while the circular column, having a pleasing appearance, may be left plain, and only needs painting.

Base and cap details will be generally as shown in Figs. 24 and 26, and floor beam connections will be generally as shown in Fig. 25.

Design 4—Lattice Girder Footbridge. 60′-0″ span, with 5′-0″ wide footway.

The general outline of the bridge, together with loads in members and sections for tubular design, with typical details, are shown in Figs. 33-37.

Calculations are as follows :—

Cross Girders.—5′-6″ span, with load of $1\frac{3}{4}$ tons, equally distributed over 5′ 0″.

B.M. $= \frac{7}{8}(33 - 15) = \frac{7}{8} \times 18 = 15{\cdot}75$ tons/ins.

As the member is supported laterally throughout its whole length, a working stress of 9 tons will be adopted.

$$Z \text{ required} = \frac{15{\cdot}75}{9} = 1{\cdot}75$$

Use $3\frac{1}{2}$″ *Bore, Class B Tube.*

Z given = 1·92

Ratio of depth to span = 66/4 = 16·5.

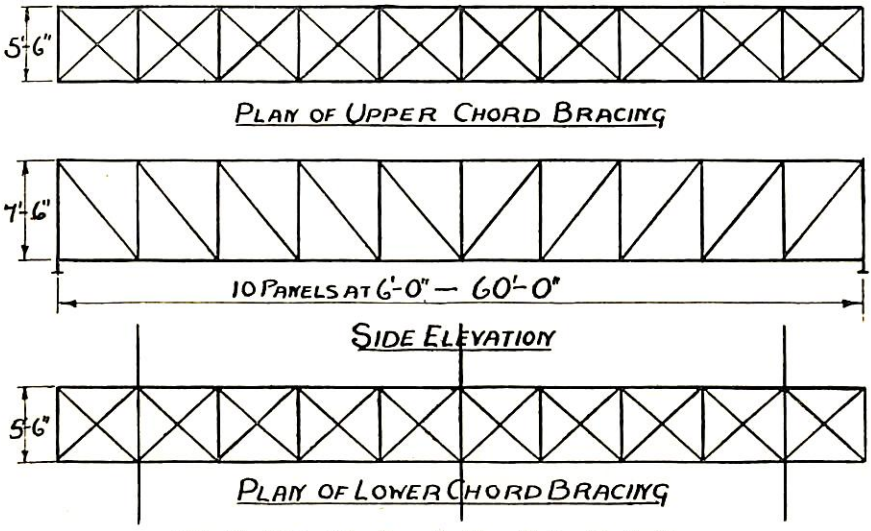


Fig. 33—60 ft.-0 in. Span Lattice Girder Footbridge.

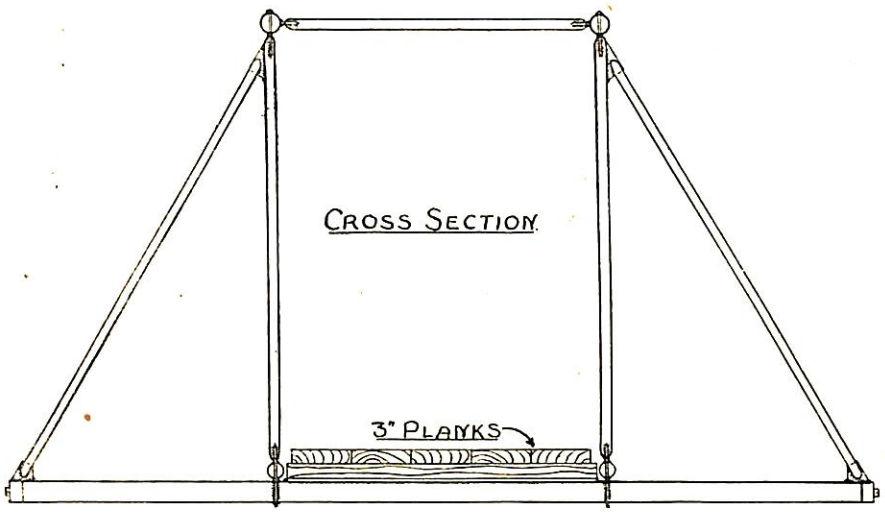
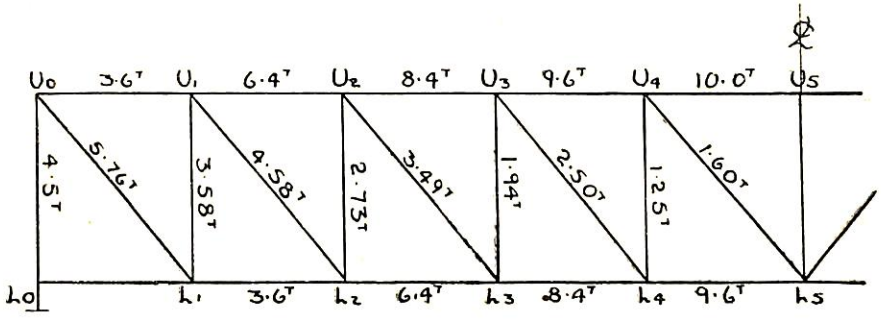


Fig. 34—60 ft.-0 in. Span Lattice Girder Footbridge.



LOADS IN MAIN GIRDER MEMBERS.

Fig. 35—60 ft.-0 in. Span Lattice Girder Footbridge.

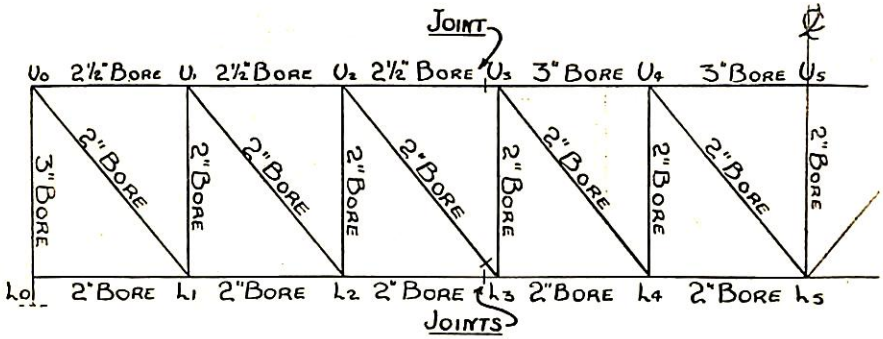


Fig. 36—60 ft.-0 in. Span Lattice Girder Footbridge. Section for Main Girders. All Class B Tubes.

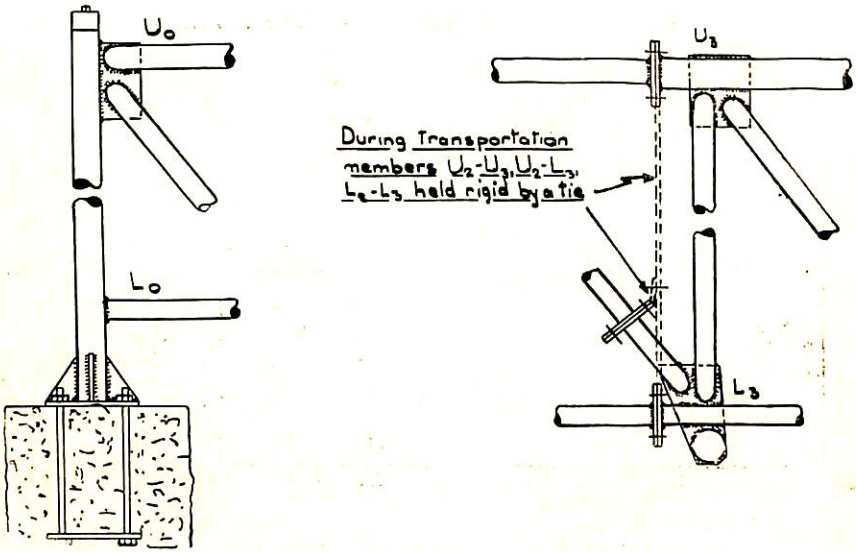


Fig. 37—60 ft.-0 in. Span Lattice Girder Footbridge. Details of Joints.

Main Girders.

LOWER CHORD—Maximum load = 9·6 tons tension.

Area of section required = $\frac{9{\cdot}6}{9}$ = 1·07 sq. ins.

Use 2" Bore, Class B Tube—Area given = 1·11 sq. ins.

This section will be maintained throughout the lower chord for construction purposes.

UPPER CHORD—Maximum load = 10·0 tons compression.

Try 3" Bore, Class B, Tube. Area given = 1·83 sq. ins.

$$\frac{L}{r} = \frac{72}{1{\cdot}18} = 61.$$

Allowable stress = 9 (1 − ·0054 × 61) = 6·03 tons.
Safe load = 6·03 × 1·83 = 11·03 tons.
Use this section for U4-U5 and U3-U4 to suit joints in chord.
For U0-U3 maximum load = 8·4 tons compression.

Try 2½" Bore, Class B Tube—Area given = 1·56 sq. ins.

$$\frac{L}{r} = \frac{72}{1} = 72$$

Allowable stress = 9 (1 − ·0054 × 72) = 5·5 tons.
Safe load = 5·5 × 1·56 = 8·58 tons.

DIAGONALS.

U0-L1. Load = 5·76 tons tension.

Area of section required = $\frac{5{\cdot}76}{9}$ = 0·64 sq. ins.

Use 2" Bore, Class B Tube for uniformity.
U1-L2 and *U2-L3*. Loads are less than in *U0-L1*, but use same section for uniformity.
U3-L4. Load = 2·5 tons tension, with reversal of 0·1 ton compression.
Compression is so small that it may be ignored, providing a margin of strength is allowed for same.
Area required for tension = 2·5/9 = 0·28 sq. ins
Use 2" Bore, Class B Tube for uniformity.
Area of section = 1·11 sq. ins, leaving an ample allowance for the compression.
U4 - L5. Load = 1·6 tons tension, with reversal of 0·8 ton compression.
Designing first on the basis of the tension giving worst condition.
Design loads (B.S.S. 153) = 1·6 tons tension + 0·4 ton compression.

Area of section = 1·11 sq. ins.

$$f_t = \frac{1{\cdot}6}{1{\cdot}11} = 1{\cdot}44 \text{ tons.}$$

$$f_c = \frac{0{\cdot}4}{1{\cdot}11} = 0{\cdot}36 \text{ ton}$$

$$f_t + f_c = 1{\cdot}80 \text{ tons.}$$

Allowable, $f_t + f_c = 9$ tons.

Designing now on the basis of the compression giving worst condition.

Design loads = 0·8 tons compression + 0·8 ton tension.

$$f_c = \frac{0{\cdot}8}{1{\cdot}11} = 0{\cdot}72 \text{ ton}$$

$$f_t = \frac{0{\cdot}8}{1{\cdot}11} = 0{\cdot}72 \text{ ton}$$

$$f_c + f_t = 1{\cdot}44 \text{ tons.}$$

$$\frac{L}{r} = \frac{102}{0{\cdot}79} = 135$$

Allowable $f_c + f_t = 9\ (1 - {\cdot}0054 \times 135) = 2{\cdot}44$ tons.

Section is therefore satisfactory.

Bracings.

Loads in these members are very small, and practically negligible. 1½″ *Bore, Class B Tube,* will be used throughout.

Joints.

The main girders are jointed near panel points U3 and L3 for convenience of transport, the members concerned being provided with flanges bolted together.

The joint in U2-U3 is in compression and in this case the bolts merely serve for tacking purposes. L2-L3 and U2-L3 are in tension, and the bolts through the flanged joints in these cases must be sufficient to withstand the tensile loads.

Tension in L2-L3 = 6·4 tons.

Using ¾″ bolts, area at bottom of thread = 0·3 sq. ins.

$$\text{With 6 bolts tensile stress} = \frac{6{\cdot}4}{6 \times 0{\cdot}3} = 3{\cdot}56 \text{ tons.}$$

This will be considered satisfactory.

For joint in U2-L3 with a load of 3·49 tons, four bolts ¾″ diameter will be provided.

Site Assembly and Welding.

In view of the small total weight of the structure the bridge may be assembled and site welded adjacent to the site, and then lifted bodily into position. The main girders are despatched in three pieces, so that the only site welding required is for the assembly of the cross girders and bracing members, which is not a difficult matter.

General Note on Designs.

It will be noted that in each design, tubes of the same type have been used throughout, that is, in the examples taken, all tubes are either Class B or Class C. This is desirable, in order to avoid confusion and possible mistakes in taking tubes from stock, as Class B and Class C tubes are of different thicknesses.

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The extracts from B.S. 153, "Girder Bridges," B.S. 449, 1948 ; The Use of Structural Steel in Buildings, B.S. 789a, "Steel Tubes and Tubulars, light-weight and heavy-weight qualities (revised weights)" and B.S. 1387—1947 ; "Steel Tubes and Tubulars suitable for screwing to B.S. 21 Pipe Threads," are given by permission of the British Standards Institution, 28 Victoria Street, London, S.W.1, from whom official copies of the specifications can be obtained ; price 2/6, 6/-, 2/6 and 2/6 each respectively, post free.

TABLE 1.

Properties of Hollow Circular Column Sections.

Outside Diam. D ins.	Inside Diam. d ins.	Thickness. t ins.	Area of Section. A ins.^{2}	Weight per foot. lbs.	Moment of Inertia. I ins.^{4}	Modulus of Section. Z ins.^{3}	Radius of Gyration. r ins.	r^2 ins.^{2}
18	15	1½	77·8	264·4	2668	296·4	5·86	34·34
18	15½	1¼	65·8	223·6	2320	257·7	5·94	35·28
18	16	1	53·4	181·6	1936	215·1	6·02	36·24
18	16½	¾	40·6	138·2	1515	168·3	6·10	37·21
18	17	½	27·5	93·5	1054	117·1	6·19	38·32
16	13	1½	68·3	232·3	1815	226·9	5·15	26·52
16	13½	1¼	57·9	196·9	1587	198·3	5·23	27·35
16	14	1	47·1	160·2	1331	166·4	5·32	28·30
16	14½	¾	35·9	122·2	1047	130·9	5·40	29·16
16	15	½	24·4	82·8	732	91·5	5·48	30·03
15	12½	1¼	54·0	183·6	1287	171·5	4·88	23·81
15	13	1	44·0	149·5	1083	144·4	4·96	24·60
15	13½	¾	33·6	114·2	855	113·9	5·05	25·50
15	14	½	22·8	77·5	600	80·0	5·13	26·32
14	11½	1¼	50·1	170·2	1027	146·7	4·53	20·52
14	12	1	40·8	138·9	868	124·0	4·61	21·25
14	12½	¾	31·2	106·1	687	98·2	4·69	22·00
14	13	½	21·2	72·1	484	69·1	4·77	22·75
13	11	1	37·7	128·2	683	105·1	4·26	18·15
13	11¼	⅞	33·3	113·3	616	94·7	4·30	18·49
13	11½	¾	28·9	98·1	543	83·6	4·34	18·84
13	11¾	⅝	24·3	82·6	466	71·7	4·38	19·18
13	12	½	19·6	66·7	384	59·1	4·42	19·54
13	12¼	⅜	14·9	50·6	296	45·5	4·46	19·89
12	10	1	34·6	117·5	527	87·8	3·91	15·29
12	10¼	⅞	30·6	104·0	476	79·3	3·95	15·60
12	10½	¾	26·5	90·1	421	70·2	3·99	15·92
12	10¾	⅝	22·3	75·9	362	60·4	4·03	16·24
12	11	½	18·1	61·4	299	49·9	4·07	16·56
12	11¼	⅜	13·7	46·6	231	38·6	4·11	16·89

TABLE 2.

Properties of Hollow Circular Column Sections.

Outside Diam. D ins.	Inside Diam. d ins.	Thickness. t ins.	Area of Section. A ins.²	Weight per foot. lbs.	Moment of Inertia. I ins.⁴	Modulus of Section. Z ins.³	Radius of Gyration. r ins.	Radius of Gyration. r² ins.²
11	9¼	⅞	27·8	94·6	359·3	65·3	3·59	12·89
11	9½	¾	24·2	82·1	318·9	58·0	3·63	13·10
11	9¾	⅝	20·4	69·3	275·1	50·0	3·67	13·47
11	10	½	16·5	56·1	227·8	41·4	3·72	13·84
11	10¼	⅜	12·5	42·6	177·0	32·2	3·76	14·14
10	8¼	⅞	25·1	85·3	263·5	52·7	3·24	10·50
10	8½	¾	21·8	74·1	234·6	46·9	3·28	10·76
10	8¾	⅝	18·4	62·6	203·1	40·6	3·32	11·02
10	9	½	14·9	50·7	168·8	33·8	3·36	11·29
10	9¼	⅜	11·4	38·6	132·0	26·4	3·41	11·63
9	7½	¾	19·4	66·1	166·7	37·1	2·93	8·58
9	7¾	⅝	16·4	55·9	145·0	32·2	2·97	8·82
9	8	½	13·4	45·4	121·0	26·9	3·01	9·06
9	8¼	⅜	10·2	34·5	94·7	21·0	3·05	9·30
8	6½	¾	17·1	58·1	113·4	28·4	2·58	6·66
8	6¾	⅝	14·5	49·2	99·2	24·8	2·62	6·86
8	7	½	11·8	40·1	83·2	20·8	2·66	7·08
8	7¼	⅜	9·0	30·5	65·4	16·4	2·70	7·29
7	5¼	⅝	12·5	42·6	64·2	18·3	2·26	5·11
7	5⅞	9/16	11·4	38·7	59·4	17·0	2·28	5·20
7	6	½	10·2	34·7	54·2	15·5	2·30	5·29
7	6⅛	7/16	9·0	30·7	48·8	13·9	2·33	5·43
7	6¼	⅜	7·8	26·5	43·0	12·3	2·35	5·53
7	6⅜	5/16	6·6	22·3	36·9	10·5	2·37	5·62
6	4⅜	⅝	10·6	35·9	38·6	12·9	1·91	3·65
6	4⅞	9/16	9·6	32·7	35·9	12·0	1·93	3·72
6	5	½	8·6	29·4	32·9	11·0	1·95	3·80
6	5⅛	7/16	7·6	26·0	29·8	9·9	1·97	3·88
6	5¼	⅜	6·6	22·5	26·3	8·8	1·99	3·96
6	5⅜	5/16	5·6	19·0	22·5	7·5	2·01	4·04

TABLE 3.

Properties of Thin Hollow Circular Column Sections.

Outside Diam. D ins.	Inside Diam. d ins.	Thickness. t ins.	Area of Section A Ins.^{2}	Weight per foot. lbs.	Moment of Inertia. I ins.^{4}	Modulus of Section. Z ins.^{3}	Radius of Gyration. r ins.	r^2 ins.^{2}
18	17¼	⅜	20·76	70·88	806	89·5	6·23	38·81
18	17⅜	5/16	17·36	59·02	678	75·3	6·25	39·06
18	17½	¼	13·94	47·40	550	61·1	6·28	39·44
16	15¼	⅜	18·41	62·59	563	70·4	5·53	30·58
16	15⅜	5/16	15·40	52·36	474	59·3	5·55	30·80
16	15½	¼	12·37	42·06	384	48·0	5·57	31·02
15	14¼	⅜	17·23	58·58	461	61·4	5·17	26·73
15	14⅜	5/16	14·42	49·03	389	51·8	5·19	26·94
15	14½	¼	11·58	39·37	315	42·1	5·22	27·24
14	13¼	⅜	16·05	54·57	373	53·3	4·82	23·23
14	13⅜	5/16	13·44	45·70	315	45·0	4·84	23·42
14	13½	¼	10·80	36·72	255	36·4	4·86	23·62
13	12⅜	5/16	12·46	42·36	251	38·6	4·49	20·16
13	12½	¼	10·01	34·03	204	31·3	4·51	20·34
12	11⅜	5/16	11·47	39·00	196	32·6	4·13	17·06
12	11½	¼	9·23	31·38	159·8	26·6	4·16	17·31
11	10⅜	5/16	10·49	35·67	148·6	27·0	3·78	14·29
11	10½	¼	8·44	28·70	121·8	22·14	3·80	14·44
10	9⅜	5/16	9·51	32·33	111·8	22·36	3·43	11·76
10	9½	¼	7·66	26·04	91·1	18·22	3·45	11·90
9	8⅜	5/16	8·53	29·00	80·4	17·85	3·07	9·42
9	8½	¼	6·87	23·36	65·6	14·58	3·09	9·55
8	7⅜	5/16	7·55	25·67	55·9	13·97	2·72	7·40
8	7½	¼	6·09	20·71	45·73	11·43	2·74	7·51
7	6½	½	5·30	18·02	30·26	8·65	2·39	5·71
6	5½	¼	4·52	15·37	18·62	6·21	2·03	4·12

TABLE 4.

Properties of Light Weight Steel Tubes.

Nom. Bore.	Outside Diam.		Thickness.		Weight per foot.	Area of Section.	Ext'nal Surface Area.	Moment of Inertia.	Modulus of Section.	Radius of of Gyration.	
	Approx.	Mean.								r	r^2
Ins.	Ins.	Ins.	S.W.G.	Ins.	Lbs.	Ins.^{2}	Ft.^{2}/Ft.	Ins.^{4}	Ins.^{3}	Ins.	Ins.^{2}
6	$6\frac{1}{2}$	6·49	8	0·160	10·835	3·182	1·699	15·947	4·914	2·24	5·01
5	$5\frac{1}{2}$	5·49	8	0·160	9·126	2·678	1·437	9·510	3·466	1·88	3·55
4	$4\frac{1}{2}$	4·48	8	0·160	7·417	2·173	1·174	5·082	2·267	1·53	2·34
$3\frac{1}{2}$	4	3·98	8	0·160	6·562	1·920	1·042	3·510	1·763	1·35	1·83
3	$3\frac{1}{2}$	3·49	8	0·160	5·708	1·673	0·913	2·320	1·331	1·18	1·38
$2\frac{1}{2}$	3	2·99	8	0·160	4·853	1·420	0·781	1·421	0·952	1·00	1·00
2	$2\frac{3}{8}$	2·37	9	0·144	3·431	1·007	0·620	0·626	0·528	0·79	0·62
$1\frac{1}{2}$	$1^{29}/_{32}$	1·90	9	0·144	2·711	0·793	0·497	0·307	0·324	0·62	0·39
$1\frac{1}{4}$	$1^{11}/_{16}$	1·67	10	0·128	2·132	0·618	0·436	0·184	0·221	0·55	0·30
1	$1^{11}/_{32}$	1·33	11	0·116	1·521	0·440	0·347	0·081	0·123	0·43	0·184
$\frac{3}{4}$	$1^{1}/_{16}$	1·05	12	0·104	1·065	0·310	0·276	0·035	0·067	0·34	0·114
$\frac{1}{2}$	$^{27}/_{32}$	0·84	13	0·092	0·739	0·215	0·219	0·015	0·036	0·266	0·071
$\frac{3}{8}$	$^{11}/_{16}$	0·67	14	0·080	0·519	0·148	0·175	0·007	0·020	0·210	0·044
$\frac{1}{4}$	$^{17}/_{32}$	0·53	15	0·072	0·353	0·104	0·139	0·003	0·011	0·164	0·027
$\frac{1}{8}$	$^{13}/_{32}$	0·40	15	0·072	0·257	0·073	0·104	0·001	0·005	0·117	0·014

TABLE 5.

Properties of Heavy Weight Steel Tubes.

Nom. Bore	Outside Diam.		Thickness.		Weight per Foot.	Area of Section.	Ext'nal Surface Area.	Moment of Inertia.	Modulus of Section	Radius of Gyration.	
	Approx.	Mean.								r	r^2
Ins.	Ins.	Ins.	S.W.G.	Ins.	Lbs.	Ins.^{2}	Ft.^{2}/Ft.	Ins.^{4}	Ins.^{3}	Ins.	Ins.^{2}
6	$6\frac{1}{2}$	6·50	6	0·192	12·936	3·806	1·702	18·953	5·831	2·23	4·98
5	$5\frac{1}{2}$	5·50	6	0·192	10·885	3·201	1·440	11·282	4·104	1·88	3·53
4	$4\frac{1}{2}$	4·49	6	0·192	8·835	2·595	1·176	6·013	2·676	1·52	2·32
$3\frac{1}{2}$	4	3·99	6	0·192	7·809	2·292	1·045	4·145	2·077	1·35	1·81
3	$3\frac{1}{2}$	3·50	6	0·192	6·784	1·994	0·916	2·735	1·564	1·17	1·37
$2\frac{1}{2}$	3	3·00	6	0·192	5·759	1·691	0·784	1·670	1·115	0·99	0·99
2	$2\frac{3}{8}$	2·38	7	0·176	4·134	1·219	0·623	0·745	0·626	0·78	0·61
$1\frac{1}{2}$	$1^{29}/_{32}$	1·91	7	0·176	3·253	0·958	0·500	0·363	0·381	0·62	0·38
$1\frac{1}{4}$	$1^{11}/_{16}$	1·68	8	0·160	2·611	0·762	0·439	0·222	0·264	0·54	0·291
1	$1^{11}/_{32}$	1·34	9	0·144	1·845	0·539	0·350	0·097	0·145	0·424	0·180
$\frac{3}{4}$	$1^{1}/_{16}$	1·06	10	0·128	1·278	0·376	0·278	0·042	0·079	0·333	0·111
$\frac{1}{2}$	$^{27}/_{32}$	0·85	11	0·116	0·902	0·266	0·222	0·018	0·043	0·262	0·068
$\frac{3}{8}$	$^{11}/_{16}$	0·68	12	0·104	0·648	0·187	0·177	0·008	0·023	0·205	0·042
$\frac{1}{4}$	$^{17}/_{32}$	0·54	13	0·092	0·431	0·129	0·141	0·003	0·012	0·161	0·026
$\frac{1}{8}$	$^{13}/_{32}$	0·40	13	0·092	0·309	0·089	0·105	0·001	0·006	0·114	0·013

TABLE 6.

Properties of Class B Tubes.

Nom. Bore	Outside Diam.		Thickness.		Weight per Foot.	Area of Section.	Ext'nal Surface Area.	Moment of Inertia.	Modulus of Section.	Radius of Gyration.	
	Approx.	Mean.								r	r^2
Ins.	Ins.	Ins.	S.W.G.	Ins.	Lbs.	Ins.^{2}	Ft.^{2}/Ft.	Ins.^{4}	Ins.^{3}	Ins.	Ins.^{2}
6	6½	6·50	7	0·176	11·891	3·497	1·702	17·490	5·381	2·236	5·000
5	5½	5·50	7	0·176	10·006	2·943	1·440	10·448	3·800	1·875	3·516
4	*4½	4·49	7	0·176	8·117	2·387	1·176	5·559	2·476	1·526	2·329
3½	4	3·99	7	0·176	7·172	2·109	1·045	3·844	1·923	1·332	1·775
3	3½	3·50	7	0·176	6·246	1·834	0·917	2·545	1·454	1·178	1·388
2½	3	3·00	7	0·176	5·301	1·559	0·785	1·563	1·042	1·001	1·003
2	2⅜	2·38	8	0·160	3·795	1·116	0·623	0·692	0·581	0·787	0·620
1½	1 29/32	1·91	8	0·160	2·988	0·876	0·500	0·340	0·356	0·673	0·388
1¼	1 11/16	1·68	9	0·144	2·357	0·694	0·440	0·206	0·245	0·545	0·297
1	1 11/32	1·34	10	0·128	1·651	0·485	0·351	0·091	0·137	0·434	0·188
¾	1 1/16	1·06	11	0·116	1·173	0·344	0·278	0·039	0·073	0·336	0·113
½	27/32	0·85	12	0·104	0·825	0·242	0·222	0·017	0·040	0·264	0·070
⅜	11/16	0·68	13	0·092	0·573	0·168	0·178	0·006	0·018	0·190	0·036
¼	17/32	0·54	14	0·080	0·391	0·115	0·141	0·003	0·011	0·161	0·026
⅛	13/32	0·40	14	0·080	0·273	0·080	0·105	0·002	0·010	0·158	0·025

TABLE 7.

Properties of Class C Tubes.

Nom. Bore.	Outside Diam.		Thickness.		Weight per Foot.	Area of Section.	Ext'nal Surface Area	Moment of Inertia.	Modulus of Section.	Radius of Gyration.	
	Approx.	Mean.								r	r^2
Ins.	Ins.	Ins.	S.W.G.	Ins.	Ins.	Ins.^{2}	Ft.^{2}/Ft.	Ins.^{4}	Ins.^{3}	Ins.	Ins.^{2}
6	$6\frac{1}{8}$	6·50	5	0·212	14·241	4·188	1·702	20·631	6·348	2·219	4·926
5	$5\frac{1}{8}$	5·50	5	0·212	11·971	3·521	1·440	12·335	4·485	1·872	3·503
4	$4\frac{1}{8}$	4·49	5	0·212	9·695	2·851	1·176	6·538	2·906	1·514	2·293
$3\frac{1}{2}$	4	3·99	5	0·212	8·557	2·515	1·045	4·505	2·252	1·338	1·791
3	$3\frac{1}{2}$	3·50	5	0·212	7·442	2·189	0·917	2·973	1·699	1·165	1·358
$2\frac{1}{2}$	3	3·00	5	0·212	6·304	1·854	0·785	1·814	1·209	0·989	0·978
2	$2\frac{3}{8}$	2·38	6	0·192	4·488	1·320	0·623	0·797	0·670	0·777	0·604
$1\frac{1}{2}$	$1^{29}/_{32}$	1·91	6	0·192	3·520	1·035	0·500	0·386	0·404	0·611	0·373
$1\frac{1}{4}$	$1^{11}/_{16}$	1·68	7	0·176	2·821	0·830	0·440	0·237	0·282	0·534	0·285
1	$1^{11}/_{32}$	1·34	8	0·160	2·009	0·591	0·351	0·104	0·155	0·419	0·176
$\frac{3}{4}$	$1^{1}/_{16}$	1·16	9	0·144	1·413	0·416	0·278	0·044	0·083	0·325	0·106
$\frac{1}{2}$	$^{27}/_{32}$	0·85	10	0·128	0·982	0·289	0·222	0·020	0·047	0·262	0·069
$\frac{3}{8}$	$^{11}/_{16}$	0·68	11	0·116	0·693	0·204	0·178	0·009	0·026	0·218	0·044
$\frac{1}{4}$	$^{17}/_{32}$	0·54	12	0·104	0·482	0·142	0·141	0·004	0·015	0·167	0·028
$\frac{1}{8}$	$^{13}/_{32}$	0·40	12	0·104	0·328	0·097	0·105	0·001	0·005	0·100	0·010

TABLE 8.

Loads on Light Weight Steel Tubes. Corresponding to Various Stresses.

Stress Tons per sq. in.	Approximate Outside Diameters—Inches.														
	13/32	17/32	11/16	27/32	1 1/16	1 11/32	1 11/16	1 29/32	2 3/8	3	3 1/2	4	4 1/2	5 1/2	6 1/2
3	0·22	0·31	0·44	0·65	0·93	1·32	1·85	2·38	3·02	4·26	5·02	5·76	6·52	8·03	9·55
4	0·29	0·42	0·59	0·86	1·24	1·76	2·47	3·17	4·03	5·68	6·69	7·68	8·69	10·7	12·7
5	0·37	0·52	0·74	1·08	1·55	2·20	3·09	3·97	5·03	7·10	8·36	9·60	10·9	13·4	15·9
6	0·44	0·62	0·89	1·29	1·86	2·64	3·71	4·76	6·04	8·52	10·0	11·5	13·0	16·1	19·1
7	0·51	0·73	1·04	1·51	2·17	3·08	4·33	5·55	7·05	9·94	11·7	13·4	15·2	18·7	22·3
8	0·59	0·83	1·18	1·72	2·48	3·52	4·95	6·35	8·05	11·4	13·4	15·4	17·4	21·4	25·5
9	0·66	0·93	1·33	1·94	2·79	3·96	5·56	7·14	9·06	12·8	15·1	17·3	19·6	24·1	28·6
10	0·73	1·04	1·48	2·15	3·10	4·40	6·18	7·93	10·1	14·2	16·7	19·2	21·7	26·8	31·8
10 2/3	0·78	1·11	1·58	2·29	3·31	4·69	6·59	8·46	10·77	15·2	17·8	20·5	23·2	28·6	33·9

TABLE 9.

Loads on Heavy Weight Steel Tubes. Corresponding to Various Stresses.

Stress Tons per sq. in.	Approximate Outside Diameters—Inches.														
	13/32	17/32	11/16	27/32	1 1/16	1 11/32	1 11/16	1 29/32	2 3/8	3	3 1/2	4	4 1/2	5 1/2	6 1/2
3	0·27	0·39	0·56	0·80	1·13	1·62	2·29	2·87	3·66	5·07	5·98	6·87	7·78	9·60	11·4
4	0·36	0·52	0·75	1·06	1·50	2·16	3·05	3·83	4·88	6·77	7·98	9·17	10·4	12·8	15·2
5	0·44	0·64	0·93	1·33	1·88	2·70	3·81	4·79	6·09	8·46	9·97	11·5	13·0	16·0	19·0
6	0·53	0·77	1·12	1·60	2·25	3·23	4·57	5·75	7·31	10·1	12·0	13·7	15·6	19·2	22·8
7	0·62	0·90	1·31	1·86	2·63	3·77	5·34	6·71	8·53	11·8	14·0	16·0	18·2	22·4	26·6
8	0·71	1·03	1·49	2·13	3·01	4·31	6·10	7·66	9·75	13·5	16·0	18·3	20·8	25·6	30·4
9	0·80	1·16	1·68	2·40	3·38	4·85	6·86	8·62	11·0	15·2	17·9	20·6	23·4	28·8	34·2
10	0·89	1·29	1·87	2·66	3·76	5·39	7·62	9·58	12·2	16·9	19·9	22·9	25·9	32·0	28·1
10 2/3	0·95	1·38	1·99	2·84	4·01	5·75	8·13	10·2	13·0	18·0	21·2	24·4	27·6	34·1	40·6

TABLE 10.

Loads on Class B Steel Tubes. Corresponding to Various Stresses.

Stress Tons per sq. in.	Approximate Outside Diameters—Inches.												
	11/16	27/32	1 1/16	1 11/32	1 11/16	1 29/32	2 3/8	3	3 1/2	4	4 1/2	5 1/2	6 1/2
3	0·50	0·73	1·03	1·46	2·08	2·63	3·35	4·68	5·5	6·3	7·2	8·8	10·5
4	0·67	0·97	1·38	1·94	2·78	3·50	4·46	6·24	7·3	8·4	9·6	11·8	14·0
5	0·84	1·21	1·72	2·43	3·47	4·38	5·58	7·80	9·2	10·6	11·9	14·7	17·5
6	1·01	1·45	2·06	2·91	4·16	5·26	6·70	9·36	11·0	12·7	14·3	17·7	21·0
7	1·18	1·71	2·41	3·40	4·86	6·13	7·81	10·92	12·8	14·8	16·7	20·6	24·5
8	1·34	1·94	2·75	3·88	5·55	7·01	8·93	12·48	14·7	16·9	19·1	23·5	28·0
9	1·51	2·18	3·10	4·37	6·25	7·88	10·04	14·04	16·5	19·0	21·5	26·5	31·5
10	1·68	2·42	3·44	4·85	6·94	8·76	11·16	15·59	18·3	21·1	23·9	29·4	35·0
10 2/3	1·79	2·58	3·66	5·17	7·40	9·34	11·90	16·63	19·5	22·5	25·5	31·4	37·3

TABLE 11.

Loads on Class C Steel Tubes. Corresponding to Various Stresses.

Stress Tons per sq. in.	Approximate Outside Diameters—Inches.												
	11/16	27/32	1 1/16	1 11/32	1 11/16	1 29/32	2 3/8	3	3 1/2	4	4 1/2	5 1/2	6 1/2
3	0·61	0·87	1·25	1·77	2·49	3·11	3·96	5·56	6·6	7·5	8·6	10·6	12·6
4	0·82	1·16	1·66	2·36	3·32	4·14	5·28	7·42	8·8	10·1	11·4	14·1	16·8
5	1·02	1·45	2·08	2·96	4·15	5·18	6·60	9·27	10·9	12·6	14·3	17·6	20·9
6	1·22	1·73	2·50	3·55	4·98	6·21	7·92	11·12	13·1	15·1	17·1	21·1	25·1
7	1·43	2·02	2·91	4·14	5·81	7·25	9·24	12·98	15·3	17·6	20·0	24·6	29·3
8	1·63	2·31	3·33	4·73	6·64	8·28	10·56	14·83	17·5	20·1	22·8	28·2	33·5
9	1·84	2·60	3·74	5·32	7·47	9·32	11·88	16·69	19·7	22·6	25·7	31·7	37·7
10	2·04	2·89	4·16	5·91	8·30	10·35	13·20	18·54	21·9	25·2	28·5	35·2	41·9
10 2/3	2·18	3·08	4·40	6·30	8·85	11·04	14·08	19·78	23·4	26·9	30·4	37·6	44·7

TABLE 12.

Permissible Stresses in M.S. Struts with Pin-Jointed Ends, for Axial Loads in Accordance with B.S.S. 153, for Girder Bridges.

Formula :— $f_c = 9 \left(1 - 0{\cdot}0054 \frac{L}{r} \right)$

Maximum Permissible Stress = 7·65 tons/sq. in.

$\frac{L}{r}$	f_c	$\frac{L}{r}$	f_c
26	7·65	74	5·40
28	7·64	76	5·31
30	7·54	78	5·21
32	7·44	80	5·11
34	7·35	82	5·01
36	7·25	84	4·92
38	7·15	86	4·82
40	7·06	88	4·72
42	6·96	90	4·63
44	6·86	92	4·53
46	6·76	94	4·43
48	6·67	96	4·33
50	6·57	98	4·24
52	6·47	100	4·14
54	6·37	102	4·04
56	6·28	104	3·95
58	6·18	106	3·85
60	6·08	108	3·75
62	5·99	110	3·65
64	5·89	112	3·56
66	5·79	114	3·46
68	5·69	116	3·36
70	5·60	118	3·26
72	5·50	120	3·17

TABLE 13.

Permissible Stresses in M.S. Struts with Rivetted Ends for Axial Loads, in accordance with B.S.S. 153 for Girder Bridges.

FORMULA, $f_c = 9\left(1 - \cdot 0038\ \frac{L}{r}\right)$.

Maximum permissible Stress = 7·65 tons per sq. inch.

$\frac{L}{r}$	f_c	$\frac{L}{r}$	f_c
38	7·65	80	6·26
40	7·63	82	6·20
42	7·56	84	6·13
44	7·50	86	6·06
46	7·43	88	5·99
48	7·36	90	5·92
50	7·29	92	5·85
52	7·22	94	5·79
54	7·15	96	5·72
56	7·08	98	5·65
58	7·02	100	5·58
60	6·95	102	5·51
62	6·88	104	5·44
64	6·81	106	5·38
66	6·74	108	5·31
68	6·67	110	5·24
70	6·61	112	5·17
72	6·54	114	5·10
74	6·47	116	5·03
76	6·40	118	4·96
78	6·33	120	4·90

TABLE 14.

Permissible Stresses in M.S. Columns for Axial Loads, and with Fixing Factor of Unity, in accordance with B.S.S. 449, for Use of Steel in Buildings.

Formula :—K2 . $Fa = \frac{fy + (n+1)\,fe}{2} - \sqrt{\left[\frac{fy + (n+1)\,fe}{2}\right]^2 - fy \,.fe}$

Maximum Permissible Stress = 9 tons per sq. inch.

$\frac{L}{r}$	Fa	$\frac{L}{r}$	Fa
10	8·51	66	5·79
12	8·42	68	5·70
14	8·32	70	5·60
16	8·23	72	5·50
18	8·13	74	5·41
20	8·03	76	5·31
22	7·93	78	5·22
24	7·83	80	5·12
26	7·73	82	5·02
28	7·63	84	4·92
30	7·54	86	4·82
32	7·44	88	4·72
34	7·35	90	4·62
36	7·25	92	4·52
38	7·16	94	4·42
40	7·06	96	4·33
42	6·96	98	4·23
44	6·86	100	4·13
46	6·76	102	4·04
48	6·67	104	3·94
50	6·57	106	3·85
52	6·47	108	3·76
54	6·38	110	3·67
56	6·28	112	3·59
58	6·19	114	3·51
60	6·09	116	3·43
62	5·99	118	3·35
64	5·89	120	3·26

TABLE 15.

Permissible Stresses in M.S. Struts for Axial Loads in accordance with the Johnson Formula.

Formula :— $f_P = f_c \left\{1 - g \left(\frac{L}{r}\right)^2\right\}$

Assumed value for $f_c = \frac{18}{2} = 9$

Assumed value for $g = 0.000057$.

Maximum stress for a very short strut = 9 tons per sq.inch.

$\frac{L}{r}$	f_c	$\frac{L}{r}$	f_c
10	8·95	46	7·92
12	8·93	48	7·82
14	8·90	50	7·72
16	8·87	52	7·61
18	8·83	54	7·50
20	8·79	56	7·39
22	8·75	58	7·27
24	8·71	60	7·15
26	8·65	62	7·03
28	8·60	64	6·90
30	8·54	66	6·77
32	8·48	68	6·63
34	8·41	70	6·49
36	8·34	72	6·35
38	8·26	74	6·20
40	8·18	76	6·04
42	8·10	78	5·88
44	8·01	80	5·72

TABLE 16—Hollow Circular Columns.

Safe Concentric Loads in tons, in accordance with B.S.S. 449—1948.

Outside diam. ins.	Thickness ins.	Wht. per foot. lbs.	Effective Heights in Feet. 50	45	40	35	30	26	22	18	14	10	8	6
18	1½	264·4	313	350	391	428	468	500	530	561	591	622	639	654
18	1¼	223·6	269	301	334	365	399	424	450	476	501	527	541	554
18	1	181·6	222	248	273	299	326	346	367	388	407	429	440	451
18	¾	138·2	171	191	210	229	250	264	280	296	311	327	335	342
18	½	93·5	118	131	144	157	170	180	191	201	211	222	227	232
16	1½	232·3	233	266	305	345	383	414	444	475	506	537	553	568
16	1¼	196·9	202	231	262	296	327	354	379	405	431	456	470	483
16	1	160·2	168	191	218	243	269	290	310	331	351	372	383	393
16	¾	122·2	130	148	168	187	207	223	238	254	269	285	292	300
16	½	82·8	90	103	115	129	142	152	163	173	183	194	199	204
15	1¼	183·6	170	197	228	260	292	318	345	369	395	422	434	448
15	1	149·5	142	164	188	214	241	261	283	303	323	344	355	365
15	¾	114·2	111	128	147	166	187	202	217	233	248	264	272	279
15	½	77·5	77	89	102	114	128	138	148	158	169	179	185	190
14	1¼	170·2	141	165	193	224	257	283	309	335	360	386	399	412
14	1	138·9	118	138	161	186	213	233	254	274	295	315	326	337
14	¾	106·1	92	108	126	144	165	180	196	211	226	242	250	258
14	½	72·1	65	75	87	100	113	123	134	144	155	165	170	176
13	1	128·2	96	114	135	158	185	205	226	247	267	288	298	308
13	⅞	113·3	86	102	121	141	165	182	201	219	237	255	264	273
13	¾	98·1	76	90	106	124	144	159	175	191	206	221	229	237
13	⅝	82·6	65	76	90	105	122	135	148	161	174	186	193	200
13	½	66·7	53	61	73	86	99	109	120	130	140	150	156	161
13	⅜	50·6	41	48	56	66	76	84	91	99	107	114	119	123
12	1	117·5		91	109	131	156	177	198	220	239	260	270	281
12	⅞	104·0		82	98	118	140	158	176	195	212	230	239	249
12	¾	90·1		72	86	103	122	138	154	169	185	200	208	216
12	⅝	75·9		62	74	88	104	117	130	143	156	169	175	182
12	½	61·4		51	61	72	85	96	106	116	127	137	142	148
12	⅜	46·6		39	47	55	65	73	81	88	96	104	108	112

For explanation of Tables of Safe Loads, see Notes under Table 21.

TABLE 17—Hollow Circular Columns.

Safe Concentric Loads, in tons, in accordance with B.S.S. 449—1948.

Outside Dia. Ins.	Thickness Ins.	Wht. per foot Lbs.	Effective Heights in Feet.											
			50	45	40	35	30	26	22	18	14	10	8	6
11	7/8	94·6			77	94	114	133	151	170	187	205	214	223
11	3/4	82·1			68	83	101	117	132	148	163	179	187	195
11	5/8	69·3			59	71	86	99	112	126	138	151	158	164
11	1/2	56·1			48	59	71	81	92	102	112	123	128	133
11	3/8	42·6			37	45	55	62	70	78	85	93	97	101
10	7/8	85·3				73	91	109	127	145	162	181	190	199
10	3/4	74·1				65	80	96	111	126	142	158	165	173
10	5/8	62·6				56	69	82	95	107	120	133	139	146
10	1/2	50·7				46	57	67	77	88	98	108	113	119
10	3/8	38·6				36	44	52	60	68	75	83	87	91
9	3/4	66·1					61	74	90	105	121	136	144	152
9	5/8	55·9					53	64	77	90	103	115	122	128
9	1/2	45·4				35	44	53	64	74	84	95	100	105
9	3/8	34·5				27	34	41	49	57	65	72	76	80
8	3/4	58·1					44	55	69	85	100	116	123	131
8	5/8	49·2					38	48	59	73	85	99	105	111
8	1/2	40·1					32	40	49	60	70	81	86	91
8	3/8	30·5					25	31	38	46	54	62	65	69
7	5/8	42·6						33	43	54	68	80	87	93
7	9/16	38·7						31	39	50	62	74	79	85
7	1/2	34·7						28	35	45	56	66	71	76
7	7/16	30·7						25	32	40	50	59	63	67
7	3/8	26·5						22	28	35	43	51	55	59
7	5/16	22·3						19	24	30	37	43	46	50
6	5/8	35·9							28	38	50	63	70	76
6	9/16	32·7							26	35	46	57	63	69
6	1/2	29·4							23	31	41	52	57	62
6	7/16	26·0							21	28	37	46	50	55
6	3/8	22·5							19	25	32	40	44	48
6	5/16	19·0							16	21	28	34	38	41

For explanation of Tables of Safe Loads, see Notes under Table 21.

TABLE 18—Thin Hollow Circular Columns.

Safe Concentric Loads, in tons, in accordance with B.S.S. 449—1948.

Outside Dia. Ins.	Thickness Ins.	Wht. per foot Lbs.	Effective Heights in Feet.											
			50	45	40	35	30	26	22	18	14	10	8	6
18	3/8	70·88	89	99	109	119	129	136	144	152	159	167	171	175
18	5/16	59·02	75	83	92	100	108	114	121	127	133	140	143	146
18	1/4	47·40	61	67	74	80	87	92	97	102	107	113	115	118
16	3/8	62·59	69	78	88	98	107	115	123	131	139	146	150	154
16	5/16	52·36	58	66	74	82	90	97	103	110	116	122	126	129
16	1/4	42·06	47	53	60	66	73	78	83	88	93	98	101	104
15	3/8	58·58	59	68	78	87	97	105	112	120	128	136	140	144
15	5/16	49·03	50	57	65	73	81	88	94	101	107	114	117	120
15	1/4	39·37	40	46	52	59	66	71	76	81	86	91	94	97
14	3/8	54·57	50	58	67	76	86	94	101	109	117	125	129	133
14	5/16	45·70	42	49	56	64	72	79	85	92	98	105	108	111
14	1/4	36·72	34	39	45	52	58	63	69	74	79	84	87	89
13	5/16	42·36	35	41	47	55	64	70	77	83	89	95	99	103
13	1/4	34·03	28	33	38	45	52	57	62	67	73	77	80	83
12	5/16	39·00		33	39	46	54	61	68	74	81	87	90	94
12	1/4	31·38		27	32	38	44	49	54	60	65	69	73	76
11	5/16	35·67			31	38	45	52	59	65	72	78	81	85
11	1/4	28·70			26	31	37	42	48	53	58	63	65	68
10	5/16	32·33			24	30	37	43	50	57	63	70	73	76
10	1/4	26·04			18	25	30	35	40	46	51	56	59	61
9	5/16	29·00				23	29	35	41	47	54	60	64	67
9	1/4	23·36				18	23	28	33	39	44	49	51	54
8	5/16	25·67					21	26	32	39	45	52	55	57
8	1/4	20·71					17	21	26	31	37	42	45	47
7	1/4	18·02						15	19	24	30	35	37	40
6	1/4	15·37							13	17	22	28	30	33

For explanation of Tables of Safe Loads, see Notes under Table 21.

TABLE 19.—Hollow Circular Columns.

Safe Concentric Loads, in tons, in accordance with Johnson's Formula, for values of Working Stress, given in Table 15.

Outside diam. Ins.	Thickness Ins.	Wht. per foot Lbs.	Effective Heights in Feet. 40	35	30	26	22	18	14	10	8	6
18	1½	264·4		494	550	587	619	646	668	683	689	695
18	1¼	223·6		422	469	499	526	547	565	578	583	587
18	1	181·6		347	383	407	428	446	459	469	473	477
18	¾	138·2	236	266	293	311	326	340	350	357	360	363
18	½	93·5	162	182	200	212	222	230	237	242	244	246
16	1½	232·3			443	486	523	553	578	596	602	608
16	1¼	196·9			380	416	446	471	491	505	510	515
16	1	160·2		273	313	341	365	384	400	412	416	420
16	¾	122·2		211	242	262	279	294	306	314	317	320
16	½	82·8		146	166	179	191	200	208	214	216	218
15	1¼	183·6			335	373	405	432	453	470	475	480
15	1	149·5			278	306	332	353	370	383	387	391
15	¾	114·2			216	237	255	271	283	293	296	299
15	½	77·5			148	162	174	184	192	199	201	203
14	1¼	170·2			287	329	363	392	416	433	439	444
14	1	138·9			239	271	298	321	339	353	358	362
14	¾	106·1			186	210	230	247	260	270	274	277
14	½	72·1			129	144	158	169	178	184	186	188
13	1	128·2				236	265	280	309	324	329	334
13	⅞	113·3				210	235	256	274	286	291	295
13	¾	98·1				184	205	223	238	249	253	256
13	⅝	82·6				156	173	188	201	210	212	215
13	½	66·7				126	140	152	162	169	172	174
13	⅜	50·6				97	107	116	123	129	131	132
12	1	117·5					231	257	279	295	300	305
12	⅞	104·0				178	205	228	246	261	266	270
12	¾	90·1				156	180	198	215	226	230	234
12	⅝	75·9				132	152	167	181	191	194	197
12	½	61·4				108	124	137	147	155	158	160
12	⅜	46·6				83	94	104	112	117	119	121

For explanation of Tables of Safe Loads, see Notes under Table 21.

TABLE 20—Hollow Circular Columns.

Safe Concentric Loads, in tons, in accordance with Johnson's Formula, for values of working stresses given in Table 15.

Outside Diam. Ins.	Thickness. Ins.	Weight per foot. Lbs.	Effective Heights in Feet. 22	18	14	10	8	6
11	7/8	94·6	173	199	219	234	240	245
11	3/4	82·1	153	174	191	204	209	213
11	5/8	69·3	130	147	162	172	176	180
11	1/2	56·1	106	120	131	140	143	145
11	3/8	42·6	81	91	100	106	108	110
10	7/8	85·3		167	190	207	213	218
10	3/4	74·1		148	167	181	187	191
10	5/8	62·6		126	141	153	157	161
10	1/2	50·7	87	103	115	124	127	130
10	3/8	38·6	68	80	88	95	98	100
9	3/4	66·1		121	142	158	164	169
9	5/8	55·9		103	121	134	139	143
9	1/2	45·4		85	99	110	114	117
9	3/8	34·5		66	76	85	87	89
8	3/4	58·1			117	135	142	147
8	5/8	49·2			100	115	121	125
8	1/2	40·1			82	94	98	102
8	3/8	30·5			63	72	75	78
7	5/8	42·6			78	94	101	106
7	9/16	38·7			71	86	92	97
7	1/2	34·7			64	78	83	87
7	7/16	30·7			57	69	73	77
7	3/8	26·5			50	60	63	66
7	5/16	22·3			43	51	54	56
6	5/8	35·9				74	82	88
6	9/16	32·7				67	74	80
6	1/2	29·4				61	67	71
6	7/16	26·0				54	59	63
6	3/8	22·5				47	52	55
6	5/16	19·0				40	44	47

For explanation of Tables of Safe Loads, see Notes under Table 21.

TABLE 21—Thin Hollow Circular Columns.

Safe Concentric Loads, in tons, in accordance with Johnson's Formula, for values of working stresses given in Table 15.

Outside diam. Ins.	Thickness Ins.	Wht. per foot Lbs.	Effective Heights in Feet.									
			40	35	30	26	22	18	14	10	8	6
18	3/8	70·88	124	138	151	160	168	174	179	183	184	186
18	5/16	59·02	104	116	127	134	141	146	150	153	154	156
18	1/4	47·40	84	93	102	108	113	117	120	123	124	125
16	3/8	62·59		112	126	136	145	151	157	161	163	164
16	5/16	52·36		94	106	114	121	127	131	135	136	137
16	1/4	42·06		75	85	91	97	102	105	108	109	110
15	3/8	58·58			112	123	132	140	145	150	152	153
15	5/16	49·03			94	103	111	117	122	126	127	128
15	1/4	39·37			76	83	89	94	98	101	102	103
14	3/8	54·57			99	110	120	128	134	139	141	143
14	5/16	45·70			83	92	101	107	112	117	118	119
14	1/4	36·72			67	74	80	86	90	94	95	96
13	5/16	42·36				81	90	97	103	108	110	111
13	1/4	34·03				65	72	78	83	86	88	89
12	5/16	39·00				70	79	87	93	98	100	102
12	1/4	31·38				57	64	70	75	79	81	82
11	5/16	35·67					68	77	84	89	91	93
11	1/4	28·70					55	62	68	72	73	74
10	5/16	32·33					57	66	74	80	82	83
10	1/4	26·04					46	54	60	64	66	67
9	5/16	29·00						55	63	70	72	74
9	1/4	23·36						45	51	56	58	60
8	5/16	25·67						44	53	61	63	65
8	1/4	20·71						36	43	49	51	53
7	1/4	18·02							35	41	43	45
6	1/4	15·37								33	36	38

For explanation of Tables of Safe Loads, see Notes below.

NOTES ON TABLES 16 to 21.

Tables 16, 17 and 18 give safe concentric loads on Hollow Circular Columns in accordance with Permissible Stresses as specified in B.S.S. 449, for Use of Steel in Buildings. The tables are based on a maximum permissible ratio of slenderness of 140, and a maximum stress of 9 tons per square inch. Loads to the right of the heavy line are for ratios of slenderness of less than 80, and in such cases it is recommended that the values given in Tables 19, 20 and 21, based on Johnson's Formula, should be used.

Tables 19, 20 and 21 give safe concentric loads on Hollow Circular Columns in accordance with Johnson's Formula, and for ratios of slenderness of less than 80

TABLE 22.

Safe Loads, in tons, on Light Weight Tubes as Struts, in accordance with B.S.S. 153 (for Struts with Pin-Jointed Ends).

FORMULA, $f_c = 9\left(1 - \cdot 0054\,\frac{L}{r}\right)$

MAX. PERMISSIBLE STRESS = 7.65 tons.

Nom. Bore Ins.	Wht. per foot Lbs.	EFFECTIVE LENGTH IN FEET.												
		22	20	18	16	14	12	10	8	7	6	5	4	3
6	10·835	10·4	12·1	13·8	15·4	17·0	18·7	20·4	22·0	22·9	23·7	24·3	24·3	24·3
5	9·126			9·1	10·8	12·5	14·2	15·8	17·5	18·3	19·1	19·9	20·5	20·5
4	7·417					7·9	9·6	11·3	12·9	13·7	14·5	15·4	16·2	16·6
3½	6·562						7·3	9·0	10·7	11·5	12·3	13·2	14·0	14·7
3	5·708							6·8	8·5	9·3	10·1	10·9	11·7	12·5
2½	4·853							4·5	6·2	7·1	7·8	8·7	9·5	10·3
2	3.431									3·9	4·7	5·3	6·1	6·8
1½	2·711										2·7	3·5	4·2	4·9
1¼	2·132											2·3	3·0	3·6
1	1·521												1·6	2·2

The above Table is compiled for a Maximum Ratio of Slenderness of 120.

For properties of Light Weight Tubes, see Table 4.

For Permissible Stresses for Ratio of Slenderness up to 120, see Table 12.

TABLE 23.

Safe Loads in tons on Heavy Weight Tubes as Struts, in accordance with B.S.S. 153 (for Struts with Pin-Jointed Ends).

FORMULA, $f_c = 9\left(1 - \cdot 0054\,\frac{L}{r}\right)$

MAX. PERMISSIBLE STRESS = 7·65 tons.

Nom. Bore Ins.	Wht. per foot Lbs.	EFFECTIVE LENGTH IN FEET.												
		22	20	18	16	14	12	10	8	7	6	5	4	3
6	12·936	12·4	14·3	16·3	18·4	20·4	22·4	24·3	26·3	27·2	28·3	29·1	29·1	29·1
5	10·885			10·9	12·9	14·8	16·9	18·9	20·9	21·9	22·9	23·8	24·5	24·5
4	8·835					9·4	11·4	13·4	15·5	16·4	17·4	18·5	19·4	19·9
3½	7·809						9·0	10·7	12·8	13·7	14·7	15·7	16·7	17·5
3	6·784							8·0	10·0	11·1	12·1	12·9	14·0	15·0
2½	5·759								7·2	8·3	9·2	10·2	11·2	12·3
2	4·134									4·6	5·5	6·4	7·3	8·3
1½	3·253										3·2	4·2	5·1	5·9
1¼	2·611											2.7	3·6	4·4
1	1·845												1·9	2·6

The above Table is compiled for a Maximum Ratio of Slenderness of 120.
For Properties of Heavy Weight Tubes, see Table 5.
For Permissible Stresses for Ratio of Slenderness up to 120, see Table 12.

TABLE 24.

Safe Loads in tons on Light Weight Tubes as Struts, in accordance with B.S.S. 153 (for Struts with rivetted ends).

FORMULA, $f_c = 9\left(1 - \cdot 0038\,\frac{L}{r}\right)$

MAXIMUM PERMISSIBLE STRESS = 7·65 tons.

Nom. Bore Ins.	Wht. per foot Lbs.	EFFECTIVE LENGTH IN FEET.												
		22	20	18	16	14	12	10	8	7	6	5	4	3
6	10·835	15·7	16·9	18·1	19·3	20·5	21·7	22·8	24·0	24·3	24·3	24·3	24·3	24·3
5	9·126			13·6	14·8	15·9	17·1	18·2	19·4	20·1	20·5	20·5	20·5	20·5
4	7·417					11·5	12·5	13·8	14·9	15·5	16·1	16·5	16·6	16·6
3½	6·562						10·3	11·4	12·6	13·2	13·8	14·3	14·7	14·7
3	5·708							9·2	10·4	11·0	11·6	12·3	12·7	12·8
2½	4·853							7·0	8·1	8·7	9·3	9·9	10·5	10·9
2	3·431								4·9	5·4	5·9	6·5	7·0	7·5
1½	2·711										4·0	4·6	5·1	5·6
1¼	2·132											3·2	3·7	4·2
1	1·521												2·3	2·7

The above Table is compiled for a maximum ratio of Slenderness of 120.
For Properties of Light Weight Tubes, see Table 4.
For Permissible Stresses for Ratios of Slenderness up to 120, see Table 13.

TABLE 25.

Safe Loads, in tons, on Heavy Weight Tubes as Struts in accordance with B.S.S. 153 (for Struts with Rivetted Ends).

FORMULA, $f_c = 9\left(1 - \cdot 0038\,\frac{L}{r}\right)$

MAXIMUM PERMISSIBLE STRESS = 7·65 tons.

Nom. Bore Ins.	Wht. per foot Lbs.	EFFECTIVE LENGTH IN FEET.												
		22	20	18	16	14	12	10	8	7	6	5	4	3
6	12·936	18·6	20·1	21·6	23·1	24·5	25·9	27·2	28·7	29·1	29·1	29·1	29·1	29·1
5	10·885			16·2	17·6	19·1	20·5	21·9	23·2	23·9	24·5	24·5	24·5	24·5
4	8·835					13·5	14·9	16·4	17·8	18·5	19·2	19·9	19·9	19·9
3½	7·809						12·2	13·6	15·1	15·8	16·5	17·2	17·5	17·5
3	6·784							10·9	12·4	13·1	13·8	14·5	15·2	15·3
2½	5·759							8·2	9·6	10·3	11·0	11·8	12·5	12·9
2	4·134									6·4	7·0	7·7	8·4	9·1
1½	3·253										4·8	5·5	6·1	6·7
1¼	2·611											4·0	4·5	5·1
1	1·845												2·8	3·3

The above Table is compiled for a maximum Ratio of Slenderness of 120.

For Properties of Heavy Weight Tubes, see Table 5.

For Permissible Stresses for Ratios of Slenderness up to 120, see Table 13.

TABLE 26.

Safe Loads in Tons on Class B Tubes as Struts, in accordance with B.S.S. 153 (for Struts with Pin-Jointed Ends).

FORMULA, $f_c = 9\left(1 - \cdot 0054\ \frac{L}{r}\right)$

MAX. PERMISSIBLE STRESS = 7·65 tons.

Nom. Bore Ins.	Wht. per foot Lbs.	EFFECTIVE LENGTH IN FEET.												
		22	20	18	16	14	12	10	8	7	6	5	4	3
6	11·891	11·4	13·2	15·1	16·9	18·7	20·6	22·4	24·3	25·1	26·0	26·8	26·8	26·8
5	10·006			10·0	11·9	13·8	15·6	17·3	19·2	20·0	21·0	21·9	22·5	22·5
4	8·117					8·7	10·6	12·5	14·3	15·1	16·1	17·0	17·9	18·3
3½	7·172						7·9	9·7	11·6	12·6	13·4	14·4	15·3	16·1
3	6·246							7·4	9·3	10·2	11·1	11·9	12·9	13·8
2½	5·301							5·0	6·9	7·7	8·6	9·5	10·4	11·3
2	3·795									4·3	5·1	5·9	6·8	7·6
1½	2·988										2·9	3·8	4·6	5·4
1¼	2·357											2·5	3·3	4·0
1	1·651												1·9	2·3

The above Table is compiled for a Maximum Ratio of Slenderness of 120.

For Properties of Class B Tubes, see Table 6.

For Permissible Stresses for Ratios of Slenderness up to 120, see Table 12

TABLE 27.

Safe Loads in Tons on Class C Tubes as Struts, in accordance with B.S.S. 153 (for Struts with Pin-Jointed Ends).

FORMULA, $f_c = 9\left(1 - \cdot 0054\,\frac{L}{r}\right)$

MAX. PERMISSIBLE STRESS = 7.65 tons.

Nom. Bore Ins.	Wht. per foot Lbs.	EFFECTIVE LENGTH IN FEET.												
		22	20	18	16	14	12	10	8	7	6	5	4	3
6	14·241	13·5	15·7	17·9	20·1	22·4	24·6	26·7	29·0	30·0	31·1	32·1	32·1	32·1
5	11·971			11·9	14·1	16·3	18·5	20·7	22·9	24·0	25·2	26·2	26·9	26·9
4	9·695					10·3	12·6	14·7	16·9	18·0	19·1	20·2	21·3	21·8
3½	8·557						9·5	11·7	13·9	15·0	16·1	17·3	18·3	19·3
3	7·442							8·7	11·0	12·1	13·2	14·2	15·4	16·4
2½	6·304								7·9	9·0	10·1	11·2	12·3	13·4
2	4·488									5·0	6·0	7·0	7·9	8·9
1½	3·520										3·5	4·4	5·4	6·4
1¼	2·821											3·0	3·8	4·8
1	2·009												2·0	2·8

The above Table is compiled for a Maximum Ratio of Slenderness of 120.
For Properties of Class C Tubes, see Table 7.
For Permissible Stresses for Ratio of Slenderness up to 120, see Table 12.

TABLE 28.

Safe Loads in Tons on Class B Tubes as Struts in accordance with B.S.S. 153 (for Struts with Rivetted Ends).

FORMULA, $f_c = 9\left(1 - \cdot 0038\,\frac{L}{r}\right)$

MAXIMUM PERMISSIBLE STRESS = 7·65 tons.

Nom. Bore Ins.	Wht. per foot Lbs.	EFFECTIVE LENGTH IN FEET.												
		22	20	18	16	14	12	10	8	7	6	5	4	3
6	11·891	17·4	18·7	19·8	21·2	22·5	23·7	25·1	26·4	26·8	26·8	26·8	26·8	26·8
5	10·006			15·1	16·2	17·5	18·7	20·0	21·3	22·0	22·5	22·5	22·5	22·5
4	8·117					12·5	13·8	15·1	16·4	17·1	17·7	18·3	18·3	18·3
3½	7·172						11·2	12·5	13·8	14·5	15·1	15·7	16·2	16·2
3	6·246							10·1	11·4	12·1	12·7	13·3	13·9	14·0
2½	5·301							7·6	8·9	9·6	10·2	10·8	11·5	11·9
2	3·795									6·0	6·6	7·1	7·7	8·3
1½	2·988										4·4	5·0	5·6	6·2
1¼	2·357											3·6	4·2	4·7
1	1·651												2·5	3·0

The above Table is compiled for a Maximum Ratio of Slenderness of 120.
For Properties of Class B Tubes, see Table 6.
For Permissible Stresses for Ratios of Slenderness up to 120, see Table 13.

TABLE 29.

Safe Loads in Tons on Class C Tubes as Struts, in accordance with B.S.S. 153 (for Struts with Rivetted Ends).

FORMULA, $f_c = 9\left(1 - \cdot 0038\,\frac{L}{r}\right)$

MAXIMUM PERMISSIBLE STRESS = 7·65 tons.

Nom. Bore Ins.	Wht. per foot Lbs.	EFFECTIVE LENGTH IN FEET.												
		22	20	18	16	14	12	10	8	7	6	5	4	3
6	14·241	20·6	22·2	23·8	25·3	26·9	28·4	30·0	31·5	32·0	32·0	32·0	32·0	32·0
5	11·971			17·8	19·4	20·9	22·5	24·0	25·5	26·3	26·9	26·9	26·9	26·9
4	9·695					14·9	16·4	18·0	19·5	20·3	21·0	21·8	21·8	21·8
3½	8·557						13·4	14·9	16·5	17·2	18·0	18·8	19·2	19·2
3	7·442							12·0	13·6	14·3	15·1	15·9	16·6	16·8
2½	6·304								10·5	11·3	12·1	12·9	13·6	14·2
2	4·488									7·0	7·7	8·4	9·1	9·8
1½	3·520										5·1	5·8	6·6	7·2
1¼	2·821											4·3	4·9	5·6
1	2·009												3·0	3·6

The above Table is compiled for a Maximum Ratio of Slenderness of 120.
For Properties of Class C Tubes, see Table 7.
For Permissible Stresses for Ratios of Slenderness up to 120, see Table 13.

TABLE 30.

British Standard Pipe Flanges. Recommended for Structural Work.

Nominal Bore of of Pipe. Ins.	Outside Diameter of Pipe. Ins.	Diameter of Flange, Ins.	Diameter of Bolt Circle. Ins.	No. of Bolts.	Diameter of Bolts. Ins.	Thickness of Flange. Ins.
2	2⅜	6	4½	4	⅝	5/16
2½	3	6½	5	4	⅝	5/16
3	3½	7¼	5¾	4	⅝	⅜
3½	4	8	6½	4	⅝	⅜
4	4½	8½	7	4	⅝	⅜
5	5½	10	8¼	8	⅝	½
6	6½	11	9¼	8	⅝	½
7	7½	12	10¼	8	⅝	½
8	8½	13¼	11½	8	⅝	½
9	9½	14½	12¾	8	⅝	⅝
10	10½	16	14	8	¾	⅝
12	12½	18	16	12	¾	⅝
14	15	20¾	18½	12	⅞	⅝
15	16	21¾	19½	12	⅞	⅝
16	17	22¾	20½	12	⅞	⅝
18	19	25¼	23	12	⅞	¾

Note.—The above sizes are normally used for Water Pipes, with a working pressure of 50-130 lbs. per square inch.

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1. Safe Load on Machine-Cut Spur Gears.
2. Deflection of Shafts and Beams } Connected.
3. Deflection of Shafts and Beams (Instruction Sheet) }
4. Steam Radiation Heating Chart.
5. Horse-Power of Leather Belts, etc.
6. Automatic Brakes (Axle Brakes) } Connected.
7. Automobile Brakes (Transmission Brakes) }
8. Capacities of Bucket Elevators.
9. Valley Angle Chart for Hoppers and Chutes.
10. Shafts up to 5½-in. diameter, subjected to Twisting and Combined Bending and Twisting.
11. Shafts, 5¾ to 26 inch diameter, subjected to Twisting and Combined Bending and Twisting.
12. Ship Derrick Booms.
13. Spiral Springs (Diameter of Rd. or Sq. Wire).
14. Spiral Springs (Compression).
15. Automobile Clutches (Cone Clutches).
16. ,, ,, (Plane Clutches).
17. Coil Friction for Belts, etc.
18. Internal Expanding Brakes. Self-Balancing Brake Shoes (Force Diagram) } Connected.
19. Internal Expanding Brakes. Angular Proportions for Self-Balancing }
20. Referred Mean Pressure Cut-Off, etc.
21. Particulars for Balata Belt Drives.
22. ⅞″ Square Duralumin Tubes as Struts.
23. 1″ ,, ,, ,, ,,
24. ¾″ Sq. Steel Tubes as Struts (30 ton yield).
25. ⅞″ ,, ,, ,, (30 ton yield).
26. 1″ ,, ,, ,, (30 ton yield).
27. ¾″ ,, ,, ,, (40 ton yield).
28. ⅞″ ,, ,, ,, (40 ton yield).
29. 1″ ,, ,, ,, (40 ton yield).
30. Moments of Inertia of Built-up Sections (Tables) } Connected.
31. Moments of Inertia of Built-up Sections (Instructions and Examples) }
32. Reinforced Concrete Slabs (Line Chart) } Connected.
33. Reinforced Concrete Slabs (Instructions and Examples) }
34. Capacity and Speed Chart for Troughed Band Conveyors.
35. Screw Propeller Design (Sheet 1, Diameter Chart) } Connected.
36. ,, ,, ,, (Sheet 2, Pitch Chart) }
37. ,, ,, ,, (Sheet 3, Notes and Examples) }
38. Open Coil Conical Springs.
39. Close Coil Conical Springs.
40. Trajectory Described by Belt Conveyors (Revised).
41. Metric Equivalents.
42. Useful Conversion Factors.
43. Torsion of Non-Circular Shafts.
44. Railway Vehicles on Curves.
45. Chart of R.S. Angle Purlins.
46. Coned Plate Developments.
47. Solution of Triangles (Sheet 1, Right Angles).
48. Solution of Triangles (Sheet 2, Oblique Angles).
49. Relation between Length, Linear Movement and Angular Movement of Lever (Diagram and Notes).
50. ,, ,, ,, ,, ,, ,, (Chart).
51. Helix Angle and Efficiency of Screws and Worms.
52. Approximate Radius of Gyration of Various Sections.

53. Helical Spring Graphs (Round Wire) } Connected.
54. ,, ,, ,, (Round Wire) }
55. ,, ,, ,, (Square Wire) }
56. Relative Value of Welds to Rivets.
57. Ratio of Length/Depth of Girders for Stiffness.
58. Graphs for Strength of Rectangular Flat Plates of Uniform Thickness.
59. Graphs for Deflection of Rectangular Flat Plates of Uniform Thickness.
60. Moment of Resistance of Reinforced Concrete Beams.
61. Deflection of Leaf Spring.
62. Strength of Leaf Spring.
63. Chart showing Relationship of Various Hardness Tests.
64. Shaft Horse Power and Proportions of Worm Gears.
65. Ring with Uniform Internal Load (Tangential Strain) } Connected.
66. Ring with Uniform Internal Load (Tangential Stress) }
67. Hub Pressed on to Steel Shaft. (Maximum Tangential Stress at Bore of Hub).
68. Hub Pressed on to Steel Shaft. (Radial Gripping Pressure between Hub and Shaft).
69. Rotating Disc (Steel) Tangential Strain } Connected.
70. ,, ,, ,, Stress }
71. Ring with Uniform External Load, Tangential Strain } Connected.
72. ,, ,, ,, ,, ,, Stress }
73. Viscosity Temperature Chart for Converting Commercial to Absolute Viscosities } Connected.
74. Journal Friction on Bearings }
75. Ring Oil Bearings }
76. Shearing and Bearing Values for High Tensile Structural Steel Shop Rivets, in accordance with B.S.S. No. 548/1934 } Connected.
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78. Velocity of Flow in Pipes for a Given Delivery } Connected.
79. Delivery of Water in Pipes for a Given Head }
80. (See No. 105).
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83. Variation of Suction Lift and Temperature for Centrifugal Pumps.
84. Nomograph for Uniformily Distributed Loads on British Standard Beams }
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86. ,, ,, ,, ,, ,, } Connected.
87. Notes on Beam Design and on Use of Data Sheets, Nos. 84-5-6. }
88. ,, ,, ,, ,, ,, }
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93-4-5-6-7. Rolled Steel Sections.
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102. Pressure Required for Blanking and Piercing.
103. Punch and Die Clearances for Blanking and Piercing.
104. Nomograph for Valley Angles of Hoppers and Chutes.
105. Permissible Working Stresses in Mild Steel Struts with B.S. 449, 1948.

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